1959 CONVENTION—LOS ANGELES—FEBRUARY 23-26, 1959

DECEMBER 1958
No. 6
VOL. 30

# JOURNAL
## OF THE
# AMERICAN CONCRETE INSTITUTE

PRINTED IN TWO PARTS—PART ONE

**Papers and Reports**.....669-740

55-42 **Continuous Reinforcement in Highway Pavements**.....
SUBCOMMITTEE VII, ACI COMMITTEE 325 669

55-43 **Stress Distribution Affects Ultimate Tensile Strength**.....
JAMES S. BLACKMAN, GERALD M. SMITH, and LYLE E. YOUNG 679

55-44 **Lightweight Concrete Deck for Tappan Zee Bridge Main Spans**.....W. G. MULLEN 685

55-45 **Shear, Diagonal Tension, and Anchorage in Beams**.....E. M. RENSAA 695

55-46 **Strains in Beams Having Diagonal Cracks**.....
D. WATSTEIN and R. G. MATHEY 717

55-47 **Influence Lines for Pressure Distribution Under a Finite Beam on Elastic Foundation**.K. C. RAY 729

**Current Reviews**.....741-748

**News Letter**..... 1-40

ACI PROCEEDINGS VOL. 55

# Detailing Practices

## Buildings to Bridges

**58 Illustrative Drawings**

The ***Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-57)*** incorporates under one cover the previous separate editions on detailing of buildings and highway structures. It is a correlation of the latest improved methods and standards for preparing drawings for the fabrication and placing of reinforcing steel. Sections on detailing and fabricating shop practice are translated into practical examples in typical drawings. Spiral bound to lie flat, it is the only publication of its kind in English and is meeting wide acclaim among designers, draftsmen, and engineering schools.

**Full Price $4.00** **ACI Members $2.50**

AMERICAN CONCRETE INSTITUTE

**PUBLICATIONS**

P.O. Box 4754, Redford Station Detroit 19, Mich.

DECEMBER 1958
Part 1

No. 6

Vol. 30

PROCEEDINGS VOL. 55

# JOURNAL
## OF THE
# AMERICAN CONCRETE INSTITUTE

**Published by the American Concrete Institute under the authority of its Board of Direction**

**President**
DOUGLAS McHENRY

**Vice-Presidents**
PHIL M. FERGUSON
JOE W. KELLY

**Directors**
ARSHAM AMIRIKIAN
WILLIAM H. ARMSTRONG
S. J. CHAMBERLIN
CLAYTON L. DAVIS
BRUCE E. FOSTER
ROBERT C. JOHNSON
BRYANT MATHER
RAYMOND C. REESE
LEWIS H. TUTHILL
C. D. WAILES, JR.
STANTON WALKER
CEDRIC WILLSON

**Past Presidents**
CHARLES S. WHITNEY
FRANK KEREKES
WALTER H. PRICE

**Secretary-Treasurer**
WILLIAM A. MAPLES

**Managing Editor**
ROBERT E. WILDE

**Advertising Manager**
CHARLES L. COUSINS

**Manager, Publication Sales**
ISABELLE DAVIS

**Membership Secretary**
EVELYN SCHLITZ

**Cost per copy $1.50.** Extra copies to ACI members $1.00.

**Papers, committee reports,** and other contributions to the **Proceedings** pages of this Journal, as submitted in furtherance of Institute objectives, are, if published, accepted on the sole responsibility of their authors. Institute authority attaches only to standards adopted as provided in the Bylaws.

**Advertising.** See statement under "Alphabetical List of Advertisers," final page **News Letter section.**

**Membership dues and Journal subscription rates.** Members receive the Journal as one of the considerations of dues payment. Membership grades are: Individual (U. S., its possessions, and North and Central America), $15, Individual (all other foreign countries), $12, Corporation, $50, Contributing, $100, Junior, $7.50, Student, $5. Subscription price is $18 per year, payable in advance. The Journal is issued monthly, July to June (the volume year being completed by title page, contents, indexes, and closing discussion in Part 2 of the subsequent September and December issues).

**Bound volumes** 1 to 53 of PROCEEDINGS of the AMERICAN CONCRETE INSTITUTE (1905 to 1957) are for sale as far as available at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

**Address:** American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich. **Copyright©, 1958,** American Concrete Institute. Printed in U. S. A. Entered in the Post Office at Detroit, Mich., as mail of the second class under provisions of the Act of Mar. 3, 1879.

## the ACI JOURNAL

**is edited by the Secretary of the Technical Activities Committee under the direction of the Committee**

WALTER J. McCOY
**Chairman**

DOUGLAS McHENRY
**(ex officio)**

WILLIAM A. MAPLES
**Secretary**

G. E. BURNETT

E. A. FINNEY

CLYDE E. KESLER

BRYANT MATHER

MICHAEL V. PREGNOFF

L. H. TUTHILL

---

WILLIAM A. MAPLES
**Editor**

ROBERT E. WILDE
**Managing Editor**

MARY K. HURD
**Associate Editor**

ROBERT G. WIEDYKE
**Associate Editor**

MARIAN PATTEN
**Editorial Assistant**

RUTH GABLE
**Editorial Secretary**


# *Coming next month in the* JOURNAL

A. M. HAAS opens the January JOURNAL lineup of technical offerings with his paper, "Concrete Space Structures—Relation Between Form and Structural Design," which he presented at the Design Session of the ACI Detroit Regional Meeting, October 27-29.

•

Coauthors EDWARD A. ABDUN-NUR and LEWIS H. TUTHILL present a method of writing realistic up-to-date specifications in their paper on "Criteria for Modern Specifications and Control."

•

RAYMOND J. SCHUTZ discusses factors influencing setting time and describes methods of control in his paper, "Setting Time of Concrete," which will appear in the January issue.

•

"Flexural Bond Tests of Prestressed Pretensioned Beams" prepared by NORMAN W. HANSON and PAUL H. KAAR presents detailed results of an investigation conducted by the authors to clarify the factors affecting bond action and strength, and to study the influence of certain variables on strand slip.

•

B. G. SINGH reports on a series of tests made to determine the "Effect of Aggregate Grading on Air Entrainment in Concrete." Paper includes tables covering nine gradings.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

Secretary, Technical Activities Committee
AMERICAN CONCRETE INSTITUTE
P. O. BOX 4754, REDFORD STATION, DETROIT 19, MICHIGAN

Title No. 55-42

**Subcommittee VII, ACI Committee 325 presents . . .**

# Continuous Reinforcement in Highway Pavements

. . . a brief description of the continuously reinforced pavements which are now under observation and their performance, a statement of the present status of committee knowledge on the subject of continuous reinforcement, recommendations for future research, and a statement of future plans of the committee.

J. D. Lindsay
**Chairman**

Henry Aaron
J. H. Moore
G. S. Paxson
William Van Breemen
W. R. Woolley*

## INTRODUCTION

■ During the past 20 years there has been an increasing interest among highway engineers in continuously reinforced concrete pavement. This interest undoubtedly developed from a dissatisfaction with the apparent deficiencies and structural weaknesses of joints in pavements, and failure, until recently, to adequately improve the design of joints and thereby eliminate their recognized defects.

Over the past 40 years pavement design has passed through a variety of phases; plain concrete pavement without joints, plain concrete pavement with expansion joints, plain concrete pavement with both expansion and contraction joints, and plain concrete pavement with contraction joints only; and reinforced concrete pavement with similar variations in the use of joints. In addition, there have been wide differences in the spacing of joints, varying from 15 ft to as much as 1000 ft.

Title No. 55-42 is a part of copyrighted Journal of the American Concrete Institute, V. 30, No. 6, Dec. 1958, *Proceedings* V. 55. Separate prints are available at 50 cents each. **Discussion** (copies in triplicate) should reach the Institute not later than Mar. 1, 1959. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.
Prepared as a part of the work of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports, and presented at the 54th annual convention, Chicago, Ill., Feb. 26, 1958. The report was submitted to letter ballot of the main committee which consists of 15 members; 11 members returned their ballots, of whom 10 voted affirmatively, and 1 negatively.
*Deceased.

**Subcommittee VII of ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports, has the assignment to study the structural design of continuously reinforced concrete pavements. J. D. Lindsay, engineer of materials, Illinois Division of Highways, Springfield, is chairman of the subcommittee. The committee included Henry Aaron, chief engineer, Wire Reinforcement Institute, Washington, D. C.; J. H. Moore, professor of civil engineering, Pennsylvania State College, State College, Pa.; G. S. Paxson, assistant state highway engineer, Oregon State Highway Department, Salem; William Van Breemen, supervising engineer, New Jersey State Highway Department, Trenton; and the late W. R. Woolley, who was associated with the Truscon Steel Division of the Republic Steel Corp., Youngstown, Ohio.**

Joints of various designs have been used with varying degrees of success but they have always had obvious shortcomings. No matter what steps were taken to improve them, joints continued to be points of potential structural weakness. They also presented problems of maintenance and permitted the infiltration of water and soil. Often they adversely affected the riding quality of pavement.

It is not meant to imply, however, that the pavement designs used during the past 40 years have been wholly unsatisfactory or that no improvements have been made. As a matter of fact, many thousands of miles of pavement built during this period have given satisfactory service.

The recent practice of cutting joints with an abrasive wheel after the concrete has hardened has eliminated some of the more serious defects attributable to joints. But the fact that joints had introduced problems which were not readily solved caused engineers to think about building continuously reinforced pavements, thereby eliminating closely spaced joints.

This interest has become increasingly active during the past 20 years, but the idea of continuous reinforcement was conceived much earlier. The first recorded attempts at continuous reinforcement were slabs 200 and 350 ft long, either 18 or 20 ft wide, constructed in 1921 by the Bureau of Public Roads on the Columbia Pike. In the early 1920's the Illinois Division of Highways constructed a long continuously reinforced pavement slab over a peat bog. There may have been other isolated attempts throughout the country but the next example of record was the experimental pavement constructed in 1938 by the Indiana State Highway Commission in cooperation with the U. S. Bureau of Public Roads, in which one test section 1310 ft long was reinforced continuously with steel bars from end to end.

Perhaps it would be well to define the term "continuously reinforced pavement." In conventionally reinforced pavements it is general practice to install transverse joints at intervals ranging from 40 to 100 ft with the rein-

forcement interrupted at the joints. Slabs of these lengths are restrained little, if any, by the subgrade and respond freely to temperature changes which tend to increase or decrease the slab length.

It has been found, however, from observations of longer slabs with larger amounts of reinforcing steel installed continuously from end to end, that length changes due to seasonal temperature cycles are reflected only within a limited distance from the ends of the slab and that the central portion remains fully restrained. Experience indicates that the length of the active portion at the end is some 400 to 500 ft. It seems logical, therefore, to define a continuously reinforced pavement as one of such length and with adequate reinforcement continuously placed, that a considerable central portion is fully restrained against longitudinal movements due to seasonal temperature changes. If one arbitrarily states that the length of the restrained central portion shall be equal to at least that of the end portions which respond to seasonal temperature changes, then one could say that a continuously reinforced pavement is one in which the length of individual slabs is not less than 1200 to 1500 ft. This length is of course a minimum limiting value and, theoretically at least, there would be no maximum limitation on length. Based on this definition, the pavements in some of the early experiments referred to above would hardly be classified as continuously reinforced. However, the behavior of these early pavements indicated the possibilities of long continuously reinforced slabs and later encouraged engineers to explore this type of construction further.

No further activity occurred in this field until 1947, when stimulated by good reports from the Indiana project and by an excellent paper on the subject by Woolley,* then of the Chicago division office of the Bureau of Public Roads, New Jersey constructed a 2 mile experimental project and Illinois built an experimental continuously reinforced pavement 5½ miles long. These were followed by a 1-mile section built by California in 1949. In 1951 three contract sections of the Fort Worth, Texas, Freeway were constructed with continuous reinforcement and additional sections were built in 1955 and 1957. During 1956 and 1957 the Pennsylvania Department of Highways constructed two experimental pavements of this type. These projects will be discussed later in this report. The highway departments of Maryland, Virginia, Connecticut, and Michigan are known to be seriously considering the construction of experimental projects to study this type of pavement.

Because of the increasing interest in this subject and the apparent successful performance of pavements now under observation, ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports, last year organized Subcommittee VII, Continuously Reinforced Pavements. This is the first report of the subcommittee. It is the purpose of this report

*Woolley, W. R., "Continuously Reinforced Concrete Pavements Without Joints." Paper first appeared in 1946, was later published in *Proceedings*, Highway Research Board, V. 27, 1947, pp. 28-33.

to give: (1) a brief description of the continuously reinforced pavements now under observation and a discussion of their performance; (2) a statement of the present status of our knowledge on the subject of continuous reinforcement; (3) recommendations for future research; and (4) a statement of future plans of the subcommittee.

## PAVEMENTS NOW IN SERVICE

Continuously reinforced pavements are now in service in Indiana, Illinois, New Jersey, California, Texas, and Pennsylvania. Except for the projects in Texas, which were constructed on the Fort Worth Expressway, all these pavements were experimental and for the purpose of obtaining information about continuous reinforcement. Most of them incorporated special research features for measuring stresses, crack widths, horizontal and vertical movements of the pavement, and other variables which are related to pavement behavior, and they have been subject to periodic surveys. The more important design features of the several pavements and the average crack interval for each are summarized in Table 1.

The Indiana project covered a wide range of reinforcing practices, and only one section would qualify under the definition of continuous reinforced pavement given above. This section is 1310 ft long and contains 1.82 percent of longitudinal steel. The cross section is 9-7-9, and it was constructed directly on a pumping type subgrade. After 15 years service it was reported to be generally in very good condition.

It will be noted that the Illinois project contains more variables than any of the others, being 7 and 8 in. thick with 0.3, 0.5, 0.7, and 1.0 percent of longitudinal steel being used with each thickness. Furthermore the pavement was constructed on the natural subgrade, 90 percent of which is composed of potentially pumping type soils. The test sections vary from 3500 to 4200 ft. After 10 years the pavement is in good condition except for a few localized structural failures at the expansion joints and at a few of the construction joints in the 0.3 and 0.5 percent sections. These failures are of little significance in the evaluation of the performance of this pavement since expansion joints would not be included in a normal pavement of this type and the weakness of construction joints can be eliminated by a change in design.

The New Jersey pavement consists of two sections, each approximately 1 mile long, one 10 in. thick and the other 8 in. thick. The pavement is on a granular subbase. A unique feature of this pavement is that it contains two layers of welded wire fabric, one 2 in. below the top surface and the other 3 in. above the bottom. The longitudinal reinforcement amounted to 0.72 percent in the 10-in. section and 0.90 percent in the 8-in. section. Except for three localized failures, involving a total of about 65 sq yd, and rupturing of the concrete at one of the terminal joints, the condition of this pavement after 10 years of service is good.

TABLE 1—FEATURES OF SEVERAL CONTINUOUSLY REINFORCED PAVEMENTS

| Location | Year constructed | Pavement thickness | Percent steel | Crack interval |
|---|---|---|---|---|
| Indiana | 1938 | 9-7-9 | 1.82 | 2.3 (1957) |
| Illinois | 1947-1948 | 7″ and 8″ | 0.3<br>0.5<br>0.7<br>1.0 | 13.0 (1957)<br>7.6 (1957)<br>6.0 (1957)<br>4.9 (1957) |
| New Jersey | 1947 | 10″<br>8″ | 0.72<br>0.90 | 5.1 (1950)<br>3.7 (1950) |
| California | 1949 | 8″<br>8″ | 0.62<br>0.50 | 3.2 (1957)<br>5.2 (1953) |
| Texas | 1951<br>1955<br>1957 | 8″<br>8″<br>No information available | 0.7<br>0.7 | 4.0 (1955) |
| Pennsylvania | 1956<br>1957 | 9″<br>7″, 8″ and 9″ | 0.48<br>0.50 | 17.9 (1957)<br>14.3 (1957) |

The California pavement is 8 in. thick and 1 mile long. The amount of longitudinal steel in one half is 0.63 percent and in the other half 0.50 percent. The pavement after 8 years of service is reported to be in excellent condition and somewhat superior to the adjacent standard pavement.

The Texas projects consist of 8-in. pavement reinforced longitudinally with 0.7 percent steel and constructed on a granular subbase. The two sections constructed in 1951 are reported to be in excellent condition. No information is available concerning the condition of the pavement constructed in 1955, nor about the 1957 construction.

One of the Pennsylvania projects is about 2 miles long and consists of 9-in. pavement with 0.48 percent longitudinal steel in the form of bar mats. The other is slightly less than 2 miles long, includes 7, 8, and 9-in. pavement all reinforced longitudinally with 0.5 percent of steel. Both bar mats and wire fabric reinforcement were used in the project. Both pavements were constructed on granular subbase. These pavements have developed the closely spaced cracks which are characteristic of this type of pavement, but neither is old enough to warrant conclusions as to behavior.

## PRESENT STATUS OF KNOWLEDGE

### Causes of cracking

Experience has shown that a long section of pavement continuously reinforced from end to end with an adequate amount of steel develops many closely spaced transverse cracks, the frequency of the cracks being proportional to the amount of longitudinal steel. The reasons for the occurrence of these cracks are not clearly understood. One cause undoubtedly is the tensile stress set up as the concrete begins to shrink, due to decrease in temperature and moisture, after the concrete has hardened. When a crack develops the stress is transferred to the steel causing a slight stretching of the steel and a small amount of opening of the crack, which provides partial

stress relief. But gradually there is a further build-up of stress, and another crack occurs.

Perhaps one of the principal causes is the restraint to longitudinal warping provided by the reinforcing steel, which tends to restrain the pavement from warping due to normal temperature gradients, thus producing flexural stress. These stresses combined with load stresses quite conceivably could exceed the flexural strength of the concrete, particularly at early ages.

This could well explain the crack interval decreasing as the percentage of steel increases. It is well known that the formation of a transverse crack provides hinge action which reduces the warping stresses in its immediate vicinity. The larger the percentage of steel, the less effective the hinge, and cracks would thus tend to be produced at intervals inversely proportional to the amount of steel present.

These are thought to be the principal reasons, although there probably are others of a secondary nature. The result of the combined effect of these factors is the early occurrence of numerous transverse cracks in a long central portion of the pavement. At some distance from the ends of the pavement, the restraint begins to diminish gradually, and the interval between successive cracks becomes progressively longer outwardly to the ends.

### Crack spacing and location

Fig. 1 shows the typical crack pattern for a continuously reinforced pavement. There are relatively few cracks near the ends, as shown at the top and bottom of the figure. But in the long central portion as shown for the two quarter points and midpoint there are many closely spaced cracks.

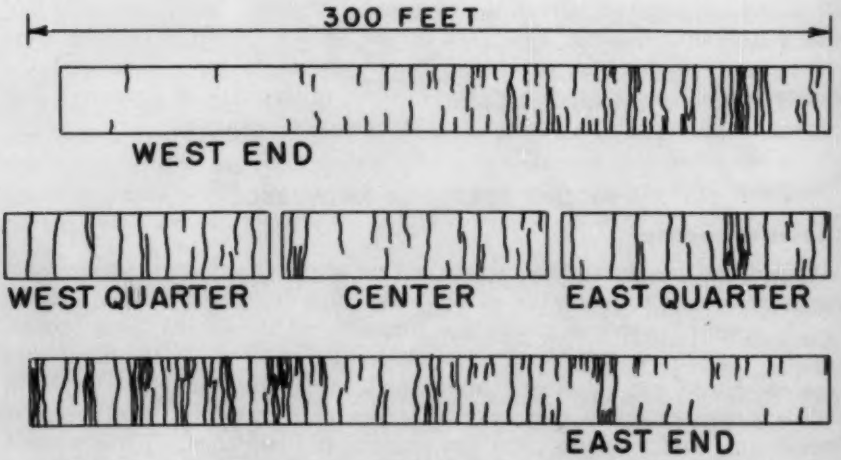



Fig. 1—Typical crack pattern for a continuously reinforced, 8-in. pavement with 0.5 percent steel

Infinitesimal width changes in the numerous cracks and elastic deformation and plastic flow of the concrete absorb the efforts of the pavement to change in length due to the daily and seasonal temperature cycles, and a long central portion remains under restraint. If this were not true, excessive movements would occur at the ends of a long pavement which no expansion joint could accommodate. Because of the restraint which is developed, however, daily movements at the ends of such a pavement are quite small, and seasonal movements are of the order of 1 in.

A large number of cracks develop at an early age, and thereafter the further development of cracks is a function of age, the rate of development decreasing rapidly with time. The average crack interval depends primarily on the amount of steel. Evidence from existing pavements indicates that very little cracking occurs after 5 or 6 years of service.

Judging from the behavior of the Illinois pavement, it appears that for 0.3 percent steel equilibrium is reached when the average crack interval is 12 to 13 ft; 8 to 9 ft for 0.5 percent steel; 6 to 7 ft for 0.7 percent and 5 to 6 ft for 1.0 percent.

### Crack width

In addition to governing the crack interval, the reinforcing steel performs another very important function; namely, it controls the amount of crack opening. It is essential to the successful performance of a continuously reinforced pavement that the steel prevent any appreciable opening of the cracks, otherwise there will be infiltration of soil and water and a loss of effective aggregate interlock. Infiltration of incompressible material into cracks in some of the experimental pavements may have been responsible for a permanent increase in their length, thus making the control of expansion at the ends a problem. It seems also that the good performance of the pavements built directly on pumpable soils can be attributed to the fact that the cracks were too narrow to permit the passage of water and soil.

It is seen then that crack interval and crack width are interrelated, and that both depend on the amount of steel. Since steel is an expensive item in pavements, the problem is to determine the optimum amount which will result in a proper balance between crack interval and crack width. In other words, it is important to determine the minimum percentage of steel that will produce the desirable characteristics which have been discussed above, namely, many closely spaced, narrow transverse cracks through which no appreciable infiltration will occur, and at which aggregate interlock will be so fully maintained that the structural integrity of the pavement will be preserved. Obviously this is a most complex problem. Although several authorities have proposed theoretical approaches, the empirical data from pavements now in service appear at present to be the most reliable. Application of these data to theoretical analyses should prove useful in the development of rational design methods.

As previously pointed out, it is essential that there be enough steel to prevent any excessive opening of the cracks. However, the accurate measurement of the width of narrow cracks is difficult. The usual practice has been to take measurements at the surface, but these measurements are likely to be influenced by raveling and warping, which result in values much greater than the true width. Cores drilled through representative cracks in the New Jersey and Illinois pavements verify this and show that, even though the cracks were quite wide at the surface, they tapered sharply to a fine crack a short distance below the surface.

A comparison of measurements taken at the surface and along the vertical edge on the Indiana project indicated that the true width was much less than the surface width. Data from the Illinois project indicate that the true widths were probably about half of the surface widths. On this basis the true widths in the Illinois project after 10 years are estimated as follows: 0.016 in. for 0.3 percent steel, 0.011 in. for 0.5 percent, 0.007 in. for 0.7 percent, and 0.005 in. for 1.0 percent steel.

Generally speaking, 0.5 percent steel appears to have given satisfactory performance from the standpoint of crack width and crack spacing. Although in the Illinois project, there was some indication of early incipient pumping at a relatively few cracks in the section of 7-in. pavement with 0.5 percent steel, this condition did not develop further. Cores taken recently through representative cracks showed a thin film of soil in the crack, and it is thought that the soil sealed the cracks and arrested the tendency to pump. There are, however, some cracks in the 0.5 percent sections that are excessively wide.

Only two of the experimental pavements were constructed without a granular subbase, these being the Indiana and the Illinois projects. Although pumping has not been a serious problem on the 1310-ft section of the Indiana project or any of the Illinois sections, except at expansion joints and construction joints, nevertheless some pumping did occur. It therefore appears desirable to use a limited thickness of subbase as a preventive measure.

### Increase in length

In one respect there was a marked difference in the behavior of the New Jersey pavement and the Illinois pavement. In particular, the New Jersey pavement showed no permanent increase in length during 10 years. Conversely, all the sections in the Illinois project after 10 years were about 4 in. longer than they were originally. The Illinois sections were separated from each other and from the adjacent pavement by 4-in. open expansion joints, but the New Jersey pavement was not afforded this amount of expansion space. Furthermore, the Illinois pavement was constructed on a subgrade of fine grained soils, whereas the New Jersey pavement was constructed on a granular subbase. It is believed that infiltration of fine soil into the transverse cracks was responsible for the permanent increase in length observed

on the Illinois sections. There is a question whether this increase would have occurred if less expansion space had been provided.

### Additional research needed

Experience obtained from the various pavements now in service indicates that it is possible to construct continuously reinforced pavements that will be exceptionally smooth riding, require little maintenance and have a long service life. However, there is not sufficient evidence to conclude that the performance of such a pavement would be outstandingly superior to that of pavements of conventional design. Furthermore, additional information is required to determine whether continuously reinforced pavements are economically feasible.

## RECOMMENDATIONS FOR FURTHER RESEARCH

Perhaps the most important need for research lies in the field of theoretical design. Several authorities have proposed design theories based on analytical concepts, and it appears desirable that these be verified with test data from actual pavements constructed in accordance with these theories. This probably should include research on bond behavior at cracks, such as progressive bond failure with time and under repetition of tensile stress and of traffic loads, as well as studies of other aspects of the mechanics involved in the interrelation between the concrete and the steel.

The relationship of pavement thickness to performance needs more study. Pavements 7 in. thick with adequate steel have given excellent service. Perhaps it would be profitable to construct limited lengths of pavements of lesser thickness, say, 5 and 6 in.

It is presently assumed that the amount of longitudinal steel should be some definite percentage of the cross sectional area of concrete. By this method of design a pavement 5 in. thick would have only half as much steel as one 10 in. thick. However, because heavy axle loads cause appreciable stress in steel reinforcement across cracks, it may be that a higher percentage of steel is needed in thin pavements than in thick pavements. The construction and observation of several experimental sections 5 or 6 in. thick with varying percentages of longitudinal steel would add greatly to our understanding of the effect of pavement thickness and percentage of steel.

The distance between transverse cracks normally varies widely. Some engineers feel that the occurrence of a series of very closely spaced cracks is detrimental. Perhaps if the location of cracks could be controlled, a better pavement would result. Research designed to find out how to control the location of transverse cracks by economical means might be worth while. It is conceivable that preforming cracks or causing cracks to develop at early ages would permit a lower percentage of longitudinal steel to be used.

## FUTURE PLANS OF THE SUBCOMMITTEE

The Committee on Rigid Pavement Design of the Highway Research Board is presently engaged in a study of continuously reinforced pavements. This work is being performed by the Subcommittee on Continuously Reinforced Concrete Pavements, several members of which are also members of Subcommittee VII of ACI Committee 325. If these two subcommittees function independently, it seems likely that there will be a certain duplication of effort. It, therefore, would be highly desirable to have close cooperation between the two groups. Perhaps their activities can be planned so as to eliminate duplication, and greater progress may result. It is suggested that the possibility of coordinating the activities of the two groups be explored.

It is understood that the Highway Research Board subcommittee has already formulated plans to make a thorough study of experimental pavements now in service and to stimulate interest in further research projects.

## ACKNOWLEDGMENTS

Subcommittee VII wishes to express its appreciation to R. K. Shaffer, Pennsylvania Department of Highways; F. N. Hveem, California Division of Highways; and W. H. Jacobs, Rail Steel Bar Association, for furnishing information for this report.

**Discussion of this report should reach ACI headquarters in triplicate by Mar. 1, 1958, for publication in the June 1959 JOURNAL.**

Specimens were subjected to combinations of axial, eccentric, and flexural loads to produce different patterns of stress distribution. The effect of stress distribution across the section has a significant effect on the ultimate tensile strength of concrete mortar specimens.

# Stress Distribution Affects ULTIMATE TENSILE STRENGTH

By James S. Blackman, Gerald M. Smith, and Lyle E. Young

Various investigators[1,2] have noted a significant difference between the axial tensile strength and the modulus of rupture of concrete. This difference may be attributable to the dissimilarity in the stress or strain distribution between axial tensile loading and flexure [Fig. 1(a) and 1(d)]. It was the purpose of this investigation to study the effect of strain distribution on the ultimate strain of concrete mortar specimens. Different stress and strain distributions were obtained on two types of specimen by applying combinations of axial, eccentric, and flexural loads.

## SPECIMENS

Specimens used were of two types: (1) Bulb end specimens combining the bulb ends of the standard tension briquet with a 1 x 1 x 6-in. prismatic midsection. (2) The second type was 1 x 1 x 11-in. prisms. All specimens were prepared from the same mix proportions:

1:2.25 by weight of 50 percent tension and 50 percent compression Ottawa sand.
$W/C$ = 0.488 by weight or 5.5 gal. per sack.

Received by the Institute July 25, 1957. Title No. 55-43 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55). Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1959. Address P. O. Box 4754, Detroit 19, Mich.

Extreme care was used in placing concrete mortar in the molds to obtain smooth true surfaces and square edges. Any specimens with visible imperfections were discarded. Concrete was placed in machined metal molds in two layers, each of which was vibrated 60 sec with a small electric vibrator. To preserve exact dimensions, specimens were struck off and troweled after subsidence had taken place. Specimens were moist cured 28 days and air dried for approximately 60 days before testing. During curing and drying periods, specimens were stored in a vertical position to minimize warping.

## TESTING METHODS

Fig. 1 shows the different stress or strain distributions obtained by various loading systems. For Loading (a), (b), and (c), bulb end specimens were used with tension applied through standard Riehle tension briquet grips. The eccentricity was obtained by off-setting the line of action of the load through the grips. For Loading (d), (e), and (f), 1 x 1 x 11-in. prisms were used. On all specimens 1-in. SR-4 strain gages were applied to opposite surfaces, i.e. those perpendicular to the plane of bending. Strains were recorded on a brush oscillograph. In all cases the gages were so placed that the strains measured were in the region of constant moment.

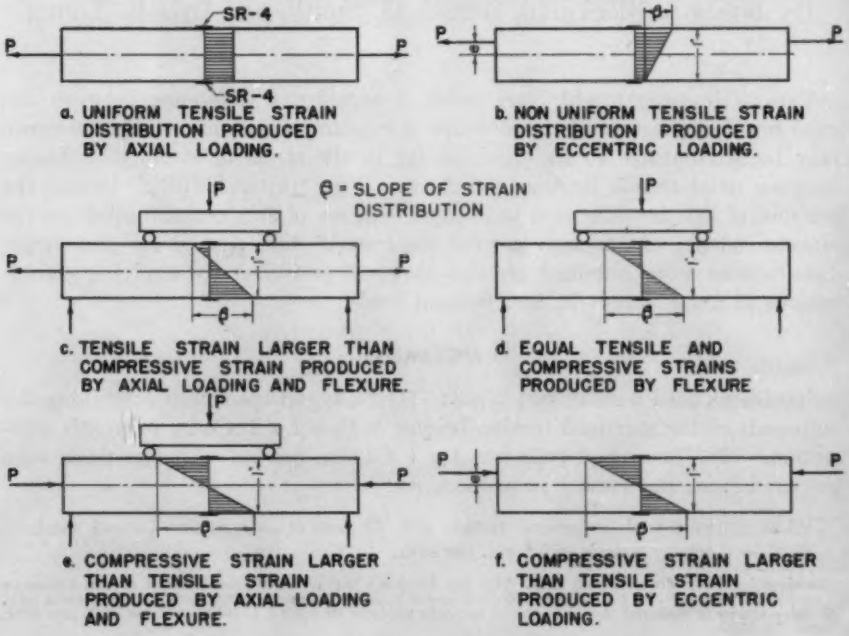



Fig. 1—Strain distribution obtained by various loading systems

Messrs. Blackman, Smith, and Young are all Institute members and colleagues in the Department of Engineering Mechanics, University of Nebraska, Lincoln, where the research they report was conducted.

JAMES S. BLACKMAN, professor of engineering mechanics, is a former field engineer for the Portland Cement Association, and is a registered professional engineer in Nebraska.

GERALD M. SMITH and LYLE E. YOUNG, both associate professors, were coauthors of two previous JOURNAL papers, "Ultimate Theory in Flexure by Exponential Function" (December 1954), and "Ultimate Flexural Analysis Based on Stress-Strain Curves of Cylinders" (December 1956). Professor Smith received the Institute's Wason Medal for 1952.

To determine the possible effects of plastic flow, a bulb end specimen was placed under axial tension at 85 percent of ultimate load and strains were recorded for a 24-hr period. The creep in 24 hr was 8 $\mu$ in., which is not of great significance considering that the principal tests were conducted in an average time interval of 15 min.

To establish the linearity of the stress-strain relation, three specimens were tested in flexure. Load, and tensile and compressive strains observed are shown in Fig. 2(a) and 2(b). Any nonlinearity in the stress-strain relationship would be apparent from these curves. Two bulb end specimens were tested in direct tension and the stress-strain relation is shown in Fig. 3. These curves are all essentially linear.

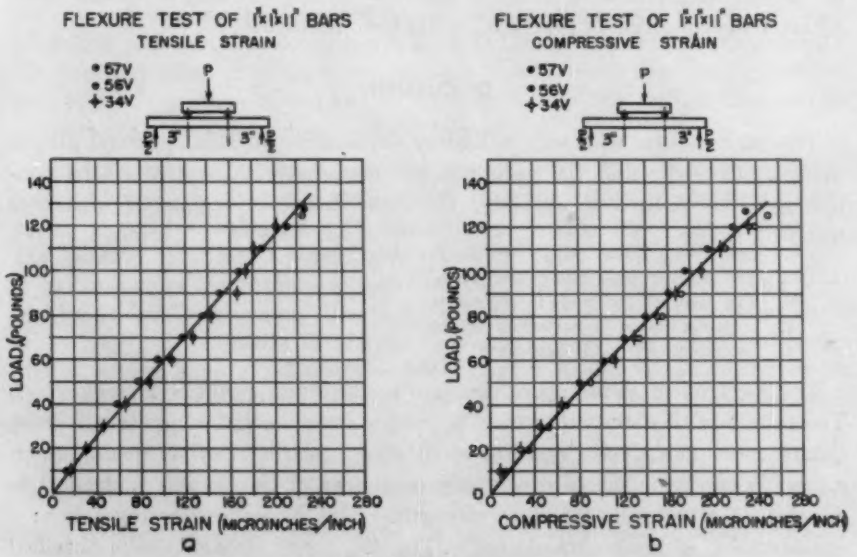



Fig. 2—Tensile and compressive strain in extreme fibers

## RESULTS

Fig. 2(a) and 3 indicate that the stress-strain relation in tension is essentially linear to failure. The modulus of elasticity in flexure was $5.5 \times 10^6$ psi, which is in good agreement with the modulus of elasticity in direct tension, $5.49 \times 10^6$ psi. From these considerations it would seem that the effects related to strain are equally applicable to stress.

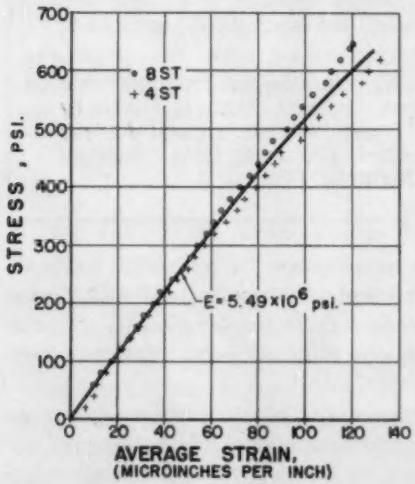



Fig. 3—Stress-strain relation for direct tension

Fig. 4 shows the effect of strain distribution on the ultimate tensile strain. The strain distribution is represented by $\beta$, which is the difference in strains between extreme fibers divided by the distance between these fibers. Eighty-three percent of the points plotted fall within the generally accepted ± 15 percent of the average curve. There is a definite trend showing an increase in ultimate tensile strain with an increase in $\beta$. The average ultimate tensile strains vary from 140 $\mu$ in. per in. to about 250 $\mu$ in. per in., an increase of about 80 percent. It appears that this trend levels off after the condition of pure flexure has been reached.

## DISCUSSION

The authors can offer only a limited explanation of this observed phenomenon. Consider first the nature of the material. Illustrative of the nonhomogeneous nature of concrete, there are localized regions of weakness resulting from:

1. The microscopic voids in the paste
2. The macroscopic voids in the mortar
3. Imperfect bond of paste and aggregate
4. Imperfections in the aggregate

The mechanism of failure of concrete under stress is not well understood. The results of this study, however, appear to indicate that concrete will resist failure more effectively, and therefore show greater relative strain, under a loading condition where a *relatively small* area of the section is stressed to a point at or near its ultimate strength. This hypothesis emerges from a consideration of the points plotted in Fig. 4. Lower strengths were obtained in direct tension, that is when the entire cross section is stressed in tension

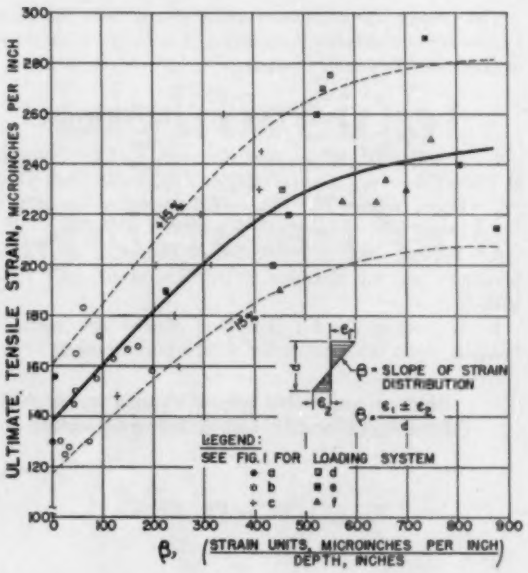



**Fig. 4—Effect of strain distribution on the ultimate tensile strain**

to near failure, than when progressively smaller areas of the cross section were stressed in tension to near failure by the device of combining bending with axial loads. The authors are at a loss to explain the mechanism associated with this behavior. However, there seems to be little doubt that the strength of concrete is related in some way to the rate of change of stress across the section. One could speculate that failure might be associated with the size and number of localized regions of weakness, i.e., the probability that localized regions of weakness occurring within the critical area are in proportion to the area stressed to near failure.[3]

The variation of ultimate tensile stress or strain with the strain distribution as observed in these tests agrees with the difference between direct tensile strength and modulus of rupture reported by other investigators.[1,2] The pattern established herein also seems to agree with the effect of size of specimens[4] on the ultimate stress or strain. The magnitude of $\beta$ (see Fig. 4) can be varied by changing the loading system as was done in these tests, or by varying the depth of the specimens. Flexure specimens with large depths would have a relatively lower value of $\beta$ and would therefore be expected to have a lower ultimate stress or strain. This is consistent with the observed results of Kellerman.[4]

## ACKNOWLEDGMENTS

This investigation was carried out in the testing laboratories of the Department of Engineering Mechanics of the University of Nebraska with the cooperation of Prof. E. J. Marmo, department chairman.

The project was conducted under grants from the George P. Abel Memorial Fund of the University of Nebraska Foundation, The Engineering Experiment Station, and the University of Nebraska Research Council.

## REFERENCES

1. Gonnerman, H. F., and Shuman, E. C., "Compression, Flexure, and Tension Tests of Plain Concrete," *Proceedings*, ASTM, V. 28, Part II, 1928, pp. 527-573.

2. Price, Walter H., "Factors Influencing Concrete Strength," ACI JOURNAL, V. 22, No. 6, Feb. 1951 (*Proceedings* V. 47), pp. 417-432.

3. Tucker, John, Jr., "Statistical Theory of the Effect of Dimensions and of Method of Loading Upon the Modulus of Rupture of Beams," *Proceedings*, ASTM, V. 41, 1941, pp. 1072-1088.

4. Kellerman, W. F., "Effect of Size of Specimens, Size of Aggregate and Method of Loading Upon the Uniformity of Flexural Strength Tests," *Public Roads*, Jan. 1933.

**Discussion of this paper should reach ACI headquarters in triplicate by Mar. 1, 1959, for publication in June 1959 JOURNAL.**

Title No. 55-44

Proportioning, testing, and field control are described in detail. The project demonstrated that high strength lightweight concrete may be produced using cement factors similar to those required for normal weight concrete through the use of drier consistencies with lowered total water content. Presaturation of aggregates and stabilization of free moisture content was the key to successful control. The contribution of natural sand content to compressive strength and workability is mentioned.

# Lightweight Concrete Deck for Tappan Zee Bridge Main Spans

By W. G. MULLEN

STRUCTURAL LIGHTWEIGHT CONCRETE was used in the roadway slab on the main spans of the Tappan Zee Bridge, the 3-mile, six-lane, Hudson River crossing of the New York State Thruway. The main cantilever bridge consists of a 1200-ft center span and two side spans of 600 ft each. It is on these spans that structural lightweight concrete was used for the 6¾-in. roadway slab and for maintenance sidewalks and curbs. An inch thick asphalt plank wearing course covers the road slab.

Concrete mix calculations were predicated on an ultimate strength at 28 days of 4000 psi at a unit weight not to exceed 110 lb per cu ft. It was decided that this could be accomplished using a structural quality expanded shale aggregate, some natural sand, entrained air, and a 6 to 7 bag per cu yd cement factor. It was felt that problems of segregation and fluctuation of yield could be avoided by proper treatment of free and absorbed water and use of a low slump. To this end, a test program was initiated.

## Test program

The test program was conducted in three parts in the laboratory followed by a field test. Final adjustments were made in the field.

In the first part, mixtures were tested containing 5, 6, 7, and 8 bags of cement per cu yd, 15 percent natural sand by absolute volume of total aggregate, 6 percent air content, 3-in. slump, and an admixture to retard set and

Received by the Institute Feb. 7, 1958. Title No. 55-44 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55). Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1959. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

TABLE 1—CHARACTERISTICS OF MATERIALS

| Item | Bulk dry specific gravity | Absorption, percent | Fineness modulus | Dry unit weight, lb per cu ft | Dry loose weight, lb per cu ft | 2-in. cube compressive strength, percent Ottawa at 28 days |
|---|---|---|---|---|---|---|
| Sand 1* | 2.63 | 0.7 | 2.59 | 106.4 | — | 111.2 |
| Sand 2* | 2.61 | 0.6 | 3.08 | 111.4 | — | 103.5 |
| Fine expanded shale | 1.75 | 5.09 | 4.56 | 59.2 | 51.6 | 64.2 |
| Coarse expanded shale | 1.50 | 6.8 | 6.88 | 43.3 | 38.3 | — |
| Cement, NYSDPW Type 2 | 3.15 | | | | | |
| Air-entraining agent—Vinsol resin | | | | | | |
| Retarder—Liquid | | | | | | |

*Sands 1 and 2 were used in separate plants on opposite sides of river.

reduce water requirements. Unit weight of fresh concrete was in the range of 105 lb per cu ft and strength results were promising for the 7-bag mix. The lightweight aggregate used in this test was from a source other than that submitted by the contractor for approval as his source of supply and had a bulk specific gravity higher than that which was used in actual construction. Tests in the second and third parts were made with the aggregate to be used in the work, a material produced from shale by a conveyor belt sintering process.

Included in the second part tests were mixtures of 6 and 7 bags per cu yd made with fine and coarse expanded shale, natural sands from two sources, New York State Department of Public Works Type 2 cement (similar to ASTM Type II), a water-reducing retarder, an air-entraining agent, and water. It was attempted to control air and slump at 6 percent and 3 in., respectively. Characteristics of materials are shown in Table 1. Tests were conducted in accordance with ASTM procedures where applicable, and air content was determined gravimetrically. From the results (Table 2 and Fig. 1) it may be seen that 4000-psi, 105-lb per cu ft concrete was obtained with a $w/c$ of 0.56 by weight and 5 percent air content.

Testing in the third part was confined to a 6.5-bag mixture in which the natural sand content was increased to 20 percent of the absolute volume of the total aggregate in an attempt to increase strength by reducing water requirements while allowing the unit weight to approach 110 lb per cu ft. Results (Table 2 and Fig. 1) were promising, but did not justify use of a lower cement factor without further testing which was impossible due to time limitations on construction.

**The work described was under the immediate supervision of W. G. MULLEN (ACI member), concrete technologist and resident engineer for Madigan-Hyland, Consulting Engineers, New York, consultants to the New York State Thruway Authority for the Tappan Zee Bridge. With the firm since 1951, Mr. Mullen, a former National Ready Mixed Concrete Association research fellow at the University of Maryland, has been concrete technologist on a number of New York State Thruway projects, including prestressed bridges for the Garden State Parkway connection, and is presently engaged on the Berkshire Thruway.**

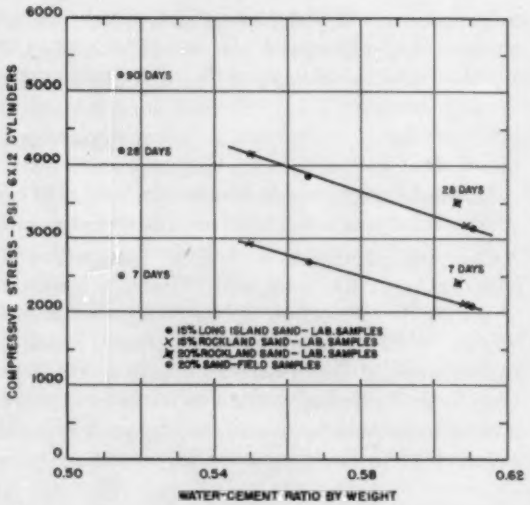



Fig. 1—Effect of water-cement ratio and natural sand content on compressive strength of expanded shale concrete

It was planned that all lightweight concrete paving would be placed in about 30 calendar days, a factor which would prevent evaluation of field results in time to make adjustments in the mix (actual placing period was 37 calendar days). Further, concreting was to be accomplished during the late fall with the anticipation of placing the bridge in service almost immediately after completion. A 7-bag cement factor was selected as the lowest safely usable in view of the nature of the test program and anticipated construction conditions.

Several days prior to start of placing operations, a 5-cu yd trial batch was made in one of the truck mixers to be used in the work. From this test batch, it was found, and later verified in actual construction, that a slump range of 2 to 2½ in. provided the most desirable consistency and that total water was reduced to 40.6 gal. per cu yd. This necessitated adjustment of mix quantities

TABLE 2—TEST MIXTURE CHARACTERISTICS AND STRENGTH RESULTS

| Mix | Cement factor, bags per cu yd | Water-cement ratio by weight | Natural sand percent of absolute volume total aggregate | Slump, in. | Air, percent | Unit weight, lb per cu ft | Compressive stress, psi | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 7 days | 28 days | 90 days |
| 1 | 6.44 | 0.610 | 15 | 3.1 | 8.1 | 101.7 | 2095 | 3147 | — |
| 2 | 7.03 | 0.565 | 15 | 3.1 | 5.2 | 105.3 | 2679 | 3859 | — |
| 3 | 6.45 | 0.608 | 15 | 3.1 | 8.0 | 101.6 | 2117 | 3196 | — |
| 4 | 7.14 | 0.549 | 15 | 2.9 | 5.0 | 105.1 | 2930 | 4126 | — |
| 5 | 6.82 | 0.606 | 20 | 3.0 | 6.2 | 104.8 | 2400 | 3508 | — |
| Field | 7.00 | 0.515 | 20 | 2.43 | 6.1 | 109.9 | 2508 | 4184 | 5220 |

Laboratory determinations are average of three rounds made on separate days with three specimens per round per variable.
Field results are average of 26 sets 6 x 12-in. cylinders with six cylinders per set tested two each at 7, 28, and, 90 days. Average 28-day result for 51 pairs of 6 x 12-in. cylinder was 4164 psi.

to maintain yield and unit weight, which was accomplished by increasing the amount of all aggregates and by adjustment of the air content to 6 percent for a unit weight of 110 lb per cu ft. Final mix proportions are included in Table 3.

## FIELD CONTROL

The lightweight concrete was the final 4400 cu yd of about 200,000 cu yd of structural concrete included in the bridge and its approaches.

Concrete inspectors were hired locally and trained in a school conducted periodically at the work site. The first group of inspectors received 2 weeks of intensive instruction in the fundamentals of concrete proportioning and control, while later groups were given 1 week of classroom instruction and a second week of field instruction with working inspectors. There were review sessions and briefing courses to cover new phases of the work. This method proved quite successful and, at the peak of construction, the inspection force included 19 working concrete inspectors.

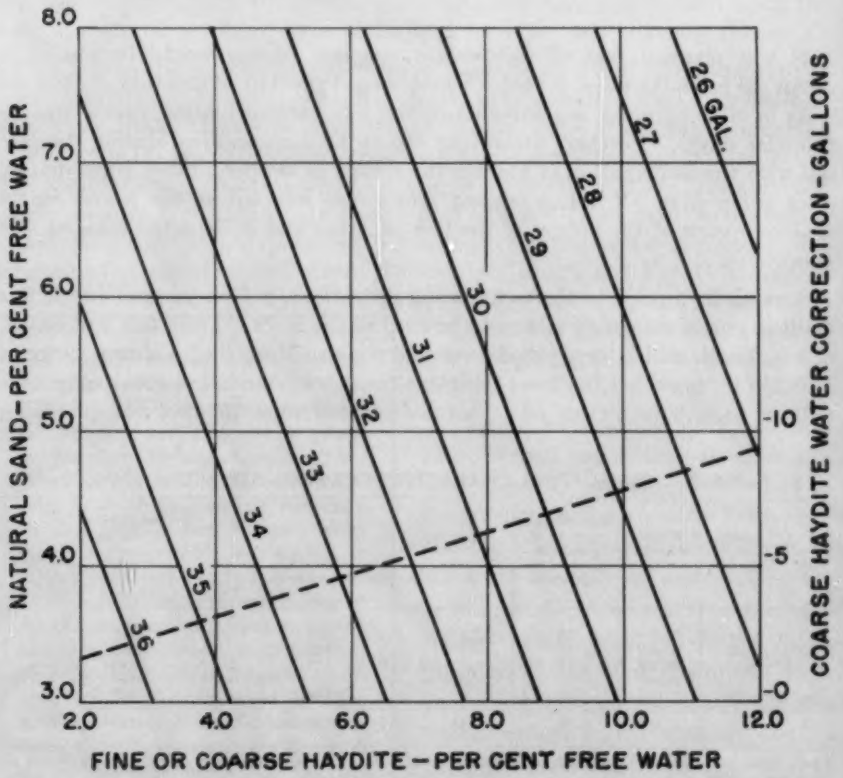



Fig. 2—Mixing water adjustment chart for 1-cu yd batch

Included in the course of instruction was a philosophy of inspection which stressed problems of personality, the need for dissemination of information, and understanding and cooperation between inspectors and contractor's personnel. Plant and field inspectors were instructed and supervised to conduct and coordinate inspection and control to allow fast, efficient concreting operations. Emphasis was placed on quantity production of high quality concrete. Instruction was pointed to the training of a competent, reasoning inspector equipped with sufficient knowledge of theory and practice to obtain quality results. It was stressed that an inspector's role is never punitive and seldom arbitrary. Intelligent coordination between plant and field inspectors and cooperation with contractor's personnel results in fast, efficient, well controlled concreting operations of benefit to contractor and owner as well. With this type inspection there will be an uninterrupted flow of concrete to the forms, provided there is sufficient equipment in use and forms are ready.

All lightweight concrete was truck mixed in 5-cu yd batches for 12 min per batch. Two batch plants were used as concrete was placed from both sides of the river to the center of the main span. One plant was an established commercial plant and the other was erected specifically for the job. There were two inspectors in each batch plant for stockpiling, batching inspection, and moisture control. Water control was positive and aggregate weights were adjusted as moisture changes occurred. Natural sand moisture was determined using the carbide type moisture meter, and moisture on both fine and coarse lightweight aggregate was determined by drying the aggregate and deducting the absorbed moisture. Moisture determinations were made before the start of work each day and then continuously through the day. As soon as one sample was completed, another was started and both sizes of lightweight aggregate were tested simultaneously.

With this running knowledge of moisture content, inspectors were able to establish and adjust batch weights and added mixing water rapidly by selecting appropriate values from a batch weight adjustment chart (Table 3 and Fig. 2). Every practical batch size for the mixing units and weighing equipment used was pre-computed in Table 3, eliminating any need by the inspectors to multiply 1-cu yd weights or to divide batches into parts for weighing. Added water chart (Fig. 2) was made up for a 1-cu yd batch and the standard 5-cu yd batch. Water to be added at any given time is selected by entering chart with free moisture of natural sand on ordinate and fine expanded shale on abcissa and picking off a gallon value from the solid diagonal lines. A deduction is found for free moisture on coarse expanded shale by entering abcissa with free moisture value, vertically intersecting dashed diagonal line, moving horizontally to right ordinate scale, and reading gallons to be deducted. For odd size batches, it was necessary to multiply a value selected from the 1-cu yd chart by the number of cubic yards in the batch.

Due to the high surface porosity of the expanded shale, it was impossible to get consistently accurate free moisture results using the carbide type moisture meter, and the hot plate method was used. Using hot plate and frying

## TABLE 3—BATCH WEIGHT ADJUSTMENT CHART

| Item | Free water | | Scale weights, lb per batch | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Percent | gal per cu yd | 1.0 cu yd | 2.0 cu yd | 2.5 cu yd | 3.0 cu yd | 4.0 cu yd | 5.0 cu yd |
| Fine aggregate | 2.0 | 1.28 | 551 | 1102 | 1378 | 1653 | 2204 | 2755 |
| | 3.0 | 1.93 | 556 | 1112 | 1390 | 1668 | 2224 | 2780 |
| | 4.0 | 2.57 | 561 | 1127 | 1403 | 1683 | 2244 | 2805 |
| | 5.0 | 3.20 | 567 | 1134 | 1418 | 1701 | 2268 | 2835 |
| | 6.0 | 3.85 | 572 | 1144 | 1430 | 1716 | 2288 | 2860 |
| | 7.0 | 4.49 | 578 | 1156 | 1445 | 1734 | 2312 | 2890 |
| | 8.0 | 5.14 | 583 | 1166 | 1456 | 1749 | 2332 | 2915 |
| Fine expanded shale | 0.0 | 0 | 767 | 1534 | 1918 | 2301 | 3068 | 3835 |
| | 2.5 | 2.19 | 785 | 1570 | 1963 | 2355 | 3140 | 3925 |
| | 5.0 | 4.38 | 804 | 1608 | 2010 | 2412 | 3216 | 4020 |
| | 7.5 | 6.57 | 822 | 1644 | 2055 | 2466 | 3288 | 4110 |
| | 10.0 | 8.76 | 840 | 1680 | 2100 | 2520 | 3360 | 4200 |
| | 12.5 | 10.95 | 858 | 1716 | 2145 | 2574 | 3432 | 4290 |
| | 15.0 | 12.14 | 876 | 1752 | 2190 | 2628 | 3504 | 4380 |
| Coarse expanded shale | 0.0 | 0 | 668 | 1336 | 1670 | 2004 | 2672 | 3340 |
| | 2.5 | 1.88 | 684 | 1368 | 1710 | 2052 | 2736 | 3420 |
| | 5.0 | 3.75 | 699 | 1398 | 1748 | 2097 | 2796 | 3495 |
| | 7.5 | 5.63 | 715 | 1430 | 1788 | 2145 | 2860 | 3575 |
| | 10.0 | 7.50 | 730 | 1460 | 1825 | 2190 | 2920 | 3650 |
| | 12.5 | 9.36 | 746 | 1492 | 1865 | 2238 | 2984 | 3730 |
| | 15.0 | 10.23 | 762 | 1524 | 1905 | 2286 | 3048 | 3810 |
| Cement—Type 2 | | | 658 | 1316 | 1645 | 1974 | 2632 | 3290 |
| Admixture—retarder | | | 14 oz | 28 oz | 35 oz | 42 oz | 56 oz | 70 oz |
| Admixture—air-entraining agent | | | 8 oz | 16 oz | 20 oz | 24 oz | 32 oz | 40 oz |

**Design weights and material constants**

| Item | cu yd—bulk dry, lb | cu yd—saturated surface dry, lb | Specific gravity | Absorption, percent | Fineness modulus | Cement factor, bbl | Remarks |
|---|---|---|---|---|---|---|---|
| Fine aggregate—sand | 535 | 540 | 2.62 | 0.7 | 2.7 | — | |
| Fine aggregate—expanded shale | 730 | 767 | 1.75 | 5.1 | 3.8 | — | ⅜ to dust |
| Coarse aggregate—expanded shale | 625 | 668 | 1.50 | 6.8 | 6.9 | — | ¾ to No. 4 |
| Cement—Type 2 | 658 | — | 3.15 | — | — | 1.75 | |
| Water | 340 | — | 1.0 | — | — | — | |
| Admixture—air-entraining agent | 8 oz | — | — | — | — | — | Vary for unit weight |
| Admixture—retarder | 14 oz | — | — | — | — | — | Liquid |

*Air content 6 percent at unit weight of 110 lb per cu ft.

pan, a moisture test could be completed on a 500-g dry weight lightweight aggregate sample in about 40 min. Water was added to trucks batched from the commercial plant by the drivers according to the determinations made above. From the other plant, water was added into the drum with the other materials according to the directions of the plant inspector. At each placing site, two inspectors supervised truck mixing of concrete, addition of admixtures, and made control tests while a third inspector supervised placing and finishing of the concrete. To say that the above system is foolproof would be a misstatement, but less than 5 percent of the batches required adjustment at the work and only ten of the 4400 cu yd placed were rejected for being overwet.

The problem posed by high absorption (Tables 1 and 2) of the expanded shale aggregates was overcome by using them in saturated surface dry plus

condition which was accomplished by use of wetted stockpiles. Specifications were so written that no aggregate could be used that had not been continuously sprinkled for 48 hr prior to use. Rigid adherence to this specification produced the desired result but with the complicating factor of free moisture contents up to 15 percent which made consistency control exacting work. After the first day of concreting, this procedure was modified so that the stockpile to be used on any given day would be sprinkled until the end of work on the day previous and then be allowed to drain over night. This was highly successful and most aggregate of both sizes was used at a stabilized free moisture between 5 and 8 percent. The crane operators loading the hoppers continually remixed the stockpiles to overcome segregation as to size and moisture content. There is no doubt that presaturation of the aggregates was the key to successful consistency control.

At the work site, admixtures were added, concrete was mixed, and unit weight and air content were controlled. Consistency was indicated by the slump test and controlled by adjusting mixing water. Air content was measured gravimetrically and controlled by varying the amount of air-entraining agent. It was noted earlier that a field test batch indicated the water requirement in the field would be several gallons less than that indicated by the laboratory tests partly due to the use of a drier consistency. Increase in all quantities to compensate for the loss of yield raised the unit weight at 4 percent air content to slightly more than 112 lb per cu ft. It was decided to place the concrete at a unit weight of 109 to 110 lb per cu ft and this was accomplished by controlling the air content between 6 and 7 percent. The expected decrease in compressive strength was offset to some extent by a lowered water-cement ratio. Unit weight determinations were made together with slump on approximately every third batch in a ½-cu ft measure on a platform scale. The amount of air-entraining agent to maintain the desired unit weight and air content varied with the temperature and other factors but was approximately 8 oz per cu yd which is about twice the amount of the same admixture needed to produce 5 percent air in a standard weight concrete. Yield control was good and actual yield varied less than 1 percent from theoretical. For the entire job, fresh unit weight averaged 109.7 lb per cu ft, slump 2.41 in., and air content 6.3 percent.

Test specimens were 6 x 12-in. cylinders made in sets of eight from the same batch at the rate of approximately one cylinder for each 10 cu yd placed. Molds were waxed cardboard with metal bottoms. Cylinders were broken in pairs at four different ages including 7 and 28 days. Average strengths for 7, 28, and 90 days were 2508, 4184, and 5220 psi, respectively. Results of all specimens tested are included in averages. Whenever possible, cylinders were moved while fresh into a heated field office and stored under damp burlap until 1 day old. Then, still in molds, they were turned upside down in a tray of water to seal the open end against evaporation loss. When cylinders could not be moved immediately, they were stored in the field overnight in an insulated box and placed in the water tray the following day. All cylinders

**Fig. 3 (left)—Concrete was placed with self-powered buggies. Fig. 4 (right)—Striking off concrete with 37-ft power screed**

were delivered to a standard laboratory moist room within 4 days, where they were cured until tested. Test specimens from some groups failed to reach 4000 psi at 28 days but always exceeded this figure at 90 days.

Some unit weight specimens were cast a foot square and 6¾ in. deep, field cured, and stored under cover at outside temperatures. After air drying for 8 months weight was 93.5 percent of weight when fresh. If behavior of concrete in service is similar, unit weight should approach 102.5 lb per cu ft.

## PLACING, FINISHING, AND CURING

Concrete was taken by power buggy from trucks or transfer hopper to the forms where it was dumped into place, consolidated by vibration, and struck off by two or more passes of a 37 ft wide power screed (Fig. 3 and 4). It is interesting to contrast the harsh dry appearance of concrete before vibration to the shiny almost wet appearance after vibration. Concrete having a slump less than 1¾ in. was too dry to be readily workable, and that having a slump in excess of 3 in. tended to creep in the forms. The ideal slump was 2 to 2½ in. and concrete was closely controlled to this range. There was no problem of segregation in handling and there was seldom any bleeding at the surface.

Following the screed machine were finishers who luted the surface using a 10-ft variable pitch aluminum float with extensible handle (Fig. 5). This tool was developed on the job to eliminate the necessity of longitudinal screeds and transverse bridges in finishing a 37 ft wide slab. Finishers worked from each side of slab lapping in the center to obtain results far better than the ⅛-in. tolerance in 10 ft specified. As the slab was to receive an asphalt plank wearing surface, no further finishing was done. Occasionally the screed machine left high spots that had to be cut by the lute. This was done easily and finisher reaction to working the surface was highly favorable. The work was described as much easier than the finishing of normal weight paving concrete.

Fig. 5—Finishers used a 10-ft lute with an extensible handle

On the curbs, where a roughened float finish was desired, it was necessary to use a cork or sponge rubber float, as a wood float did not leave the desired surface texture. Heavily reinforced curb sections were formed with excellent detail using dry consistencies and heavy vibration.

Concrete was cured 14 days under wet burlap, except where asphalt plank could be laid immediately; in those sections water cure was terminated in 7 days, surface was air dried 1 day, and asphalt primer, mastic, and plank were laid in lieu of further curing. Concreting was completed in fall and forms were not removed until spring, although removal was allowed at 14 days.

## WEARING COURSE

As stated previously, there is a 1 in. thick asphalt plank wearing course covering the structural roadway slab. A stainless steel grid approximately 12 x 12 ft was placed in the top surface of the slab to prevent creep of the asphalt plank. Originally, the contractor had planned to concrete the grid in place, but problems of securing a relatively limber grid in exact position and finishing to required surface tolerances between grid bars seemed insurmountable at the time, and it was decided to place the grid after concrete had hardened. Placing was accomplished by sawing a kerf in the concrete ¾ in. deep, wide enough for a driving fit, then hammering the grid bars home. Concrete, up to 3 days old, was sawed with an abrasive blade and, thereafter, concrete was hardened to the extent that a diamond blade was necessary. At ages less than 24 hr, concrete tended to ravel when sawed.

## SUMMARY

From the standpoint of mix selection, experience on the Tappan Zee Bridge demonstrated that high strength concrete may be produced using cement factors similar to those required for normal concrete through the use of drier consistencies with lowered total water requirement. Further, natural sand

content definitely affects strength and workability characteristics of this expanded shale concrete. It is conjectured that, with additional testing of mixes and use of control techniques described herein, still further reduction could be made in cement factor without loss of compressive strength.

Experience has affirmed that presaturation of aggregates and stabilization of free moisture content is the key to successful control. Gravimetric determination of air content is practical and accurate. Also, dry consistencies may be used together with heavy vibration without segregation of material or loss of workability.

Finally, it is noted that compressive strength of field cylinders at 90 days was about 25 percent improved over 28 days, and that unit weight loss of air dried concrete approached 7 percent.

**Discussion of this report should reach ACI headquarters in triplicate by Mar. 1, 1959, for publication in the June 1959 JOURNAL.**

Title No. 55-45

# SHEAR, DIAGONAL TENSION, AND ANCHORAGE IN BEAMS

By E. M. RENSAA

**PROBLEMS relating to shearing strength and diagonal tensile strength of reinforced concrete beams are discussed. Particular attention is given to conditions at points of contraflexure. It is pointed out that bending stresses will have an influence on both the direction of tension cracks and on the shearing strength of beams. The effect of shrinkage stresses will also have considerable influence on the diagonal tensile strength. Actual direction of cracks may cause an increase in anchorage steel tension greater than that found by ordinary bending moment theory. It is shown that the ordinary formula for shearing stresses in reinforced concrete beams is not applicable at sections where there are no bending stresses.**

SOME FAILURES OF REINFORCED CONCRETE frames in diagonal tension a few years ago focused the attention of designers on the adequacy of the shear clauses in our building codes. In spite of the considerable number of tests and theoretical investigations made during more than half a century, no general agreement has so far been reached about how to safeguard against failure in diagonal tension. An examination of building codes in North America and Europe shows a great variation of specifications for shear reinforcement.

There has, in the opinion of the writer, been too much emphasis placed on finding formulas which would be universally applicable and not enough thought given to the accuracy of fundamental assumptions for different stress conditions. The majority of textbooks show how formulas are developed without pointing out the limitations of the fundamental assumptions on which the formulas are based. The result is that most engineering students are apt to use the standard formulas with little criticism, until perhaps confronted with an unexpected mishap. Reinforced concrete is not a homogeneous material and cannot be expected to be as fitted for generalization as, for instance, structural steel.

The object of this paper is to make a contribution to a better understanding of shearing stresses and related problems in reinforced concrete. It seems possible in some cases to obtain such an understanding on the basis of theoretical considerations. In other instances this does not appear possible without additional tests.

Received by the Institute June 9, 1958. Title No. 55-45 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55). Separate prints are available at 60 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1959. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

## PROPER SHEAR

Concrete has a relatively high shearing strength and "proper shear" seldom is an initial cause of failure. It is nearly impossible to determine shearing strength accurately because of the difficulty of eliminating accompanying tensile stresses during tests. Test results, therefore, vary considerably depending on how successful the investigator has been in reducing the tensile stresses accompanying the shear. Shearing strength will, to a large extent, depend on the restraint against expansion transversely to the direction of shear. It is also likely that direct compression or tension perpendicular to the plane of shear will have considerable influence on the actual shear strength.

Tests to determine proper shearing strength of concrete have been carried out by a number of investigators. Early American tests were designed to obtain the absolute shear strengths under conditions of lateral confinement. Tests by Taylor indicated ratios of shearing strength to compressive strength of 0.44 and 0.57 depending on concrete quality. Tests by punching through plates gave ratios varying from 0.37 to 0.86 depending on the form of test piece.[1]

Tests made under the direction of Spofford on cylinders 5 in. in diameter with ends securely clamped in cylindrical bearings gave shearing-compressive strength ratios ranging from 0.63 to 1.04, the highest being for a concrete having a compressive strength of only 1110 psi.[2]

A test under conditions of what may be assumed as the greatest obtainable lateral confinement has been described by Rehm.[3] This was a pull-out test with a bar having a single shoulder which had bearing on a projection inside a hole through a concrete specimen. The maximum shearing strength was found to be approximately equal to the cube strength.

It is clear that the above test results do not represent actual conditions in practice. There is usually little or no lateral restraint acting on reinforced concrete beams and tensile stresses are not effectively eliminated. It may, therefore, be assumed that under such conditions, shearing strength to compressive strength ratios will generally be lower than the figures quoted above.

Tests by Mörsch[4] on rectangular specimens without lateral confinement indicated a shearing strength equal to $\sqrt{k_1 k_2}$ where $k_1$ is the tensile strength and $k_2$ the cube compressive strength of the concrete. Since the tensile strength will always be less than the compressive strength, the shearing strength, according to Mörsch's equation, will also be less than the compressive strength. Assuming, for instance, the tensile strength to be 8 percent of the cube strength and the standard cylinder strength to be 80 percent of the cube strength, we get:

$$\text{Shearing strength} = \sqrt{\frac{0.08 \times f_c' \times f_c'}{0.8}} = 0.316\, f_c'$$

The coefficient of 0.316 applies only to the assumed relation of tensile strength to compressive strength. Actually the tensile strength of ordinary concrete seems to vary approximately in proportion to the square root of the compressive strength, and a definite coefficient should not be assumed.

Further tests on shearing strength are reported by Suenson[5] and Rausch;[6] Laupa, Siess and Newmark[7] also list a number of references.

**E. M. RENSAA (ACI member) is a partner in the architectural-engineering firm of Rensaa & Minsos, Edmonton, Alta., Canada. Mr. Rensaa's familiarity with code problems is indicated by his membership on the Advisory Committee on Structures of the National Research Council of Canada in connection with the revision of the National Building Code. The firm of Rensaa & Minsos was instrumental in introducing precast reinforced concrete building frames into the Province of Alberta, and has designed many concrete structures.**

It seems to be clear from tests with concrete specimens having no special lateral confinement that the proper shearing strength, if it can be called such, will be influenced so largely by unavoidable tensile stresses that a direct linear relation to compressive strength does not exist. This is even more true for the diagonal tensile strength. It is, therefore, quite incorrect to specify the allowable strength in shear of unreinforced concrete to be proportional to the compressive strength, as has so far been customary in North America. It would be much more logical to stipulate a certain allowable shear for each of the standard concrete strengths.

Since there is such a wide spread between proper shearing strengths found by different investigators, it seems safest to figure with the lower values which apply for conditions of little or no lateral restraint. However, since the shearing strength in any case is much higher than the tensile strength, it is only in exceptional cases, such as where a heavy concentrated load is acting close to a support, that there is any appreciable danger of initial failure by shear. Another, and more usual case, is where a crack caused by diagonal tension has extended so far toward the compression side of the beam that the remaining uncracked concrete has insufficient shearing resistance.

## PRINCIPAL STRESSES

Even though shearing stress alone is seldom a primary cause of failure, the shear combined with bending moment tension may form a resultant diagonal tensile stress which frequently causes initial cracking. This principal stress, unless adequately reinforced against, often limits the carrying capacity of a beam. Although shear is thus only a component of the principal stress, it is generally used as a measure of the beam's ability to resist principal tensile stresses. The shear component, however, is only usable for that purpose where conditions are similar to those forming the basis for the allowable shearing stresses. Bending stresses, torsional stresses, and axial stresses will also influence the magnitude and directions of the resultant principal stresses.

Shear alone as a measure of principal stress intensity may be misleading.

This fact is unfortunately often disregarded and in the case of members subject to axial tension, the disregard may lead to unsafe designs. The adoption of the transverse shear force alone and its corresponding shearing stresses as a basis for the design of web reinforcement has caused a great deal of misunderstanding of the actual conditions of internal stresses in reinforced concrete members. It would undoubtedly have been better if, from the beginning, principal stresses had been made the basis for calculation and design of web reinforcement.

In the design of reinforced concrete beams for bending it is usual to assume that concrete has no tensile strength. This is a necessary safeguard in such cases where the tensile strength of concrete is liable to be exceeded. The assumption that concrete has no tensile strength has been used, in other instances, as a basis for the development of mathematical theories. The result in such cases may be both unrealistic and misleading. The truth is that concrete would be completely useless as a construction material if it had no tensile strength and no shear strength.

While the assumption of no tensile strength in bending is universally accepted, the practice is not so uniform when it comes to diagonal tension. European custom, in the case of beams, is either to reinforce for the complete shear on the portion of a beam where the allowable unit shearing stress for an unreinforced beam is exceeded, or to reinforce for the complete shear if the unit allowable shearing stress is exceeded on any portion of the beam. North American practice has been to assume that concrete could carry some shear and use web reinforcement for the rest. Which contention is right, the European or the North American?

Until about 1916 the rules for reinforcing against diagonal tensile stresses were about similar in Europe and North America. It was assumed that concrete would take a certain part of the shear in any case, regardless of the intensity of stress, and that it was only necessary to provide shear reinforcement for the balance of shear not taken by the concrete. This rule, which is still used in the United States, has been abandoned in Europe. The chief reason for this abandonment was information obtained from extensive tests carried out in Germany during the first quarter of this century.[8] These tests proved that the assumption of apportioning some part of the shear to the concrete would not always prevent failure in diagonal tension when a beam was loaded to its ultimate capacity. Tests were then conducted to find out what proportion of shear should be carried by the steel to prevent failure in diagonal tension. It was found that with the kind of concrete commonly used and where shear was a critical factor that it was advisable to provide web reinforcement for the complete shear. German specifications were then changed in 1916 to provide for full shear reinforcement on such portions of a beam where allowable shearing stresses without reinforcing were exceeded. During the next decade, European specifications were changed to comply with this rule which is still in general use. Further German investigations indicated that it was possible to have shear failure even on the

portion of the beam where the shearing stress was lower than 4 kg per sq cm (57 psi). In 1925 the Germans therefore again tightened up their shear specifications and prescribed full shear reinforcement for all shear if the allowable shear without reinforcing was exceeded on any part of the beam.

It might appear from the above that North American engineers should follow at least the generally accepted rule of providing full shear reinforcement where the maximum allowable shearing stress without reinforcing was exceeded. Many engineers strongly favor adoption of such a rule. However, there is a condition which should be considered before taking such a step.

The Germans have allowed the use of concrete of relatively low compressive strength. Their ordinary concrete qualities are designated as B 120, B 160, B 225, and B 300, which indicate compressive *cube* strengths in kg per sq cm at 28 days. This corresponds to cylinder strengths of about 1500, 2000, 2800, and 3400 psi, respectively. The use of the lowest strength concrete was rather common earlier and is still allowed for unimportant reinforced concrete structures. The second grade has, until recently, been more or less the standard for ordinary work. The German tests showed that for the two lower grades it was difficult to avoid crushing of concrete at the bend of bent-up bars if the steel strength was to be fully utilized and such bent-up bars are designed in German practice to take most of the shear. Safe anchorage of the smooth stirrup bars in the compression zone was also a problem, although as a rule a lesser one. Using full shear reinforcement for such low strength concrete meant in reality the lowering of the actual steel stress to such an extent that crushing of concrete and slipping of anchorage was avoided. Even Mörsch, who was the chief advocate of full shear reinforcement, found that when using the third grade, B 225, full safety against shear failure was obtained with web reinforcement for only half the shear.[9] The reason was the greater safety against crushing and anchorage failure with the higher strength concrete.

While North American codes allow the use of 2000-psi concrete, such low strength concrete is seldom used for structures of any considerable span or importance. It would be uneconomical under these circumstances, except for concrete with strength less than about 2500 psi, to specify full shear reinforcement regardless of concrete quality.

The National Building Code of Canada was changed a year ago to specify that at least two-thirds of the shear above the lower allowable limit should be taken by the web reinforcement for unit shearing stresses up to $0.06 f_c'$, above which the full shear should be taken by the web reinforcement. It is the writer's opinion that on the stretch where shear reinforcement is required it is better to reinforce for a certain proportion of the shear than to provide only for the excess shear stress block above the allowable value for concrete without web reinforcement. If, on a portion of a beam, there is any danger of formation of diagonal tension cracks, there should then be a sufficient number of stirrups at a reasonable spacing to hinder objectionable widening of the cracks. Such spacing is not feasible where the shear stress block is wedge shaped.

It has been proved by numerous tests that the remaining uncracked concrete of a section will carry some external shear. If it were possible to determine how much shear the concrete itself would carry, it would then be necessary to put in only enough web reinforcement to carry the excess shear. This reinforcement would be sufficient to safeguard the member against diagonal tension or shear failure.

To determine how much the concrete will carry is not an easy matter, however. The strength of a beam in shear depends not only on the compressive strength, shearing strength, and tensile strength of the concrete but also on the amount and distribution of the web reinforcement and even to some extent on the horizontal reinforcement. Shrinkage stresses and temperature stresses may also have a large influence. Considerable work has been done in the United States to evaluate the influence of all but the last two items and empirical formulas intending to show the ultimate shearing resistance of beams have been worked out (see for instance papers by Moretto[10] and by Laupa, Siess and Newmark[7]). It has been found that the formula by Moretto based on tests of simple beams does not apply to restrained beams.[11] According to Laupa, Siess and Newmark (Reference 7, p. 10-11) all earlier attempts have failed to establish generally applicable equations. In their paper these investigators make another attempt to work out formulas for ultimate shear strength of both rectangular and T-beams. However, the analysis is based only on beams made and tested in laboratories. The influence of longitudinal forces set up by shrinkage and temperature deformations, such as will most often be encountered in a structure where a beam is only a part of the whole, has been neglected. It is the writer's opinion that even if the equations had been suited for practical use, they would still not generally give accurate or safe values since the above mentioned additional influences have not been taken into account.

The above discussion applies to the ordinary condition of freely supported ends. Most of the tests for shear have been made on such simple beams. The principal stresses under this condition will be a combination of shearing stresses and bending stresses. Near the free end of the beam shearing stress will form the largest component while the opposite is the case toward the center of the span. There will also generally be little external axial tension from shrinkage and temperature deformation at the freely supported end of a beam. A crack in the concrete will generally start, under these conditions, from the underside of the beam and move upward toward the neutral axis. If there is no web reinforcement, the external shear will then have to be taken partly by dowel action of the bottom reinforcement and partly by the uncracked compression zone above the neutral axis.

It may be useful to consider the problem of dowel action in more detail. If a bar is embedded in a concrete body of fairly large cross section, we know that the bar will carry a considerable transverse load after the concrete has cracked. How much additional resistance the bar provides to concrete before cracking is not known by the writer but it is probably not much. In the

case of the ordinary horizontal reinforcement in a concrete beam, the bars are placed fairly close together near a surface. There will therefore be a relatively small concrete section left outside and between the bars to resist a transverse pressure from the bars. This concrete will in addition have considerable tension from transverse shrinkage which the relatively uncompressible steel bars will set up. It is not to be expected under these circumstances that the remaining small concrete section can have much dowel resistance and certainly not to an extent which will justify much dependence on it. There is still another way in which dowel action may take place, namely if a horizontal split along the bars crosses a stirrup. Such a stirrup will then take up the transverse load from the horizontal reinforcing bars. Thus, if a diagonal crack starts near a stirrup there may still be considerable help from the stirrup after a split has formed even if the diagonal crack does not cross the stirrup. This may, under certain circumstances, relieve the remaining uncracked concrete section of some of the shear load. As the stirrup gets overstressed, or the horizontal bars are deformed in bending, the shear load again transfers to the uncracked concrete section. It should also be mentioned that the portion of shear load which may be carried by the combined bar-stirrup dowel action will depend on the slope of the diagonal crack since this has an influence on the vertical offset of the adjacent sections at a crack.

In view of the above discussion it would appear to be difficult to determine how big a proportion of the shear load will be taken by dowel action and it seems safest to disregard this completely. Under these circumstances, and if there is no web reinforcement crossing the crack, the whole shear must be taken by the uncracked compression concrete. It is not likely that interlocking of aggregate across the crack will carry any appreciable portion of the shear. The contention that concrete can carry some shear is therefore true for a case where some portion is uncracked. Since concrete is relatively strong in shear, less than the complete concrete section will in many cases provide stability. However, as the crack moves upward, the section of concrete which must carry both shear and compression becomes smaller and the unit principal stress correspondingly higher. We may then get a failure caused by combined shear and compression. Thus, the function of well designed web reinforcement should be to carry some of the shear directly, and also hinder a diagonal tension crack from moving so far upward that the remaining uncracked portion becomes too small. Tests have shown that web reinforcement properly spaced and proportioned will have an effect on the shear strength of a beam even if not designed to take the full shear.

It is not easy to determine the actual distribution of shear load between concrete and web reinforcement. Where material of different kinds is used to form a structure, it is always a possibility that one kind may fail before the other gets into action. Steel will stand overstressing to a considerable extent, and in the case of web reinforcement may generally be depended on to act with the concrete until complete failure of the member takes place. If, on the other hand, the concrete should fail completely before the steel gets

into action, the whole load would, in such a case, have to be carried by the steel. This will be the case with a reinforced concrete member in tension and, as will be shown later, may also apply to a continuous beam at the point of contraflexure.

Concentrated loads and the reaction from a support have some local modifying influence on the principal stresses. Investigators have found that compression from a support increased the principal compressive stress near the support by 38 percent and decreased the principal tensile stress by 28 percent.[12] The important part of this stress change is, of course, the beneficial decrease in diagonal tension. Since this decrease only occurs near the support, it is obvious that shear of the same magnitude is more dangerous farther out on the beam.

## WHICH TYPE OF WEB REINFORCEMENT IS MOST EFFECTIVE?

In Europe the practice is to use bent-up bars as the major part of web reinforcement and space stirrups at a uniform distance to take a minor proportion of the shear. American practice favors the use of vertical stirrups and often no bars at all are bent-up. To determine what kind of web reinforcement is most effective, it is useful to study the stress trajectories of a homogeneous beam.

Fig. 1 shows the trajectories for one-half of a simply supported beam acted on by a concentrated load at the center of the span. The principal stresses will all have slopes of 45 deg at the neutral axis where there is no bending stress. However, slopes will differ considerably along the beam from the neutral axis to the tension side.

In the ordinary shear theory for reinforced concrete, we assume concrete to have no tensile strength and the weak section in diagonal tension to continue downward from the neutral axis at 45 deg. This assumption forms the basis for the Mörsch truss analogy and also for the spacing of stirrups. However, tests have proved that the assumption of concrete compression diagonals and bent-up tension steel bars forming a Warren truss with diagonals at 45 deg will not give sufficient safety against crack formation. Neither will a Pratt trust system be safe with concrete compression diagonals at 45 deg and vertical tension members consisting of stirrups. Bent-up bars should not be spaced farther apart in the longitudinal direction than a distance corresponding to the effective depth,[13] and stirrups certainly not more than 75 percent of this dimension if the web reinforcement is to be sufficiently effective. The reason for the closer spacing seems to be obvious. The average slope of a stress trajectory is steeper than 45 deg near the support and becomes almost vertical near the center of a beam. Cracks are most likely to start along a line of maximum principal tensile stress and follow such a line unless there is an influence causing a redistribution of stresses. Local weaknesses and web reinforcement crossing a crack will be parts of such a modifying influence. A truss analogy is, therefore, only of value if the compression

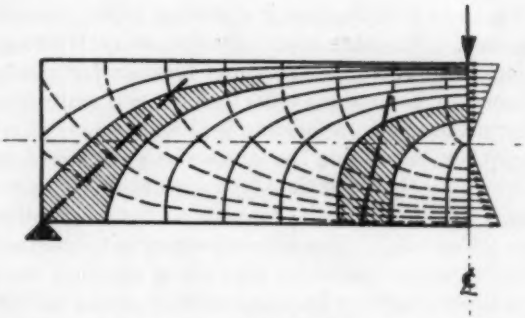


Fig. 1—Stress trajectories for simply supported beam with a concentrated load at the center of the span

diagonals have slopes corresponding to the actual slopes of compressive stresses in the beam.

Web reinforcement is supposed to be effective only when crossing a crack. If the crack is so steep that stirrups will not necessarily cross it, of how much value are the stirrups? This question is especially important for locations where both shear and bending stresses are a maximum, such as at interior supports of continuous beams and for beams having concentrated loads. An answer to this question could only be determined by tests. It would appear, however, that bent-up bars will generally provide much greater safety against diagonal cracking than can be expected from vertical stirrups. Stirrups, on the other hand, will perform the useful additional function of reinforcing a beam transversely against splitting caused by bond or anchorage stresses in the main horizontal reinforcement. An alternative to bent-up bars would be stirrups placed at a suitable slope. There can be no doubt that such stirrups, if properly anchored to the main reinforcement, would be just as effective as bent-up bars. In fact, they would be even better because of generally smaller cross section and therefore better anchorage at the compression side. Neither would there be the same danger of crushing concrete at the bends. Sloping stirrups would have the same advantage as ordinary ones in providing reinforcement in the transverse direction. That sloping stirrups are better than vertical ones in locations with both maximum bending and shear seems to be quite obvious and has been proved by tests.[14]

It should be understood that vertical stirrups will only carry the vertical component of the diagonal stress. There should be in reality a corresponding amount of horizontal reinforcement to take up the horizontal stress component of the diagonal stress.[6] If this horizontal steel is lacking, additional tension is thrown on the main horizontal reinforcement. If the beam should be relatively deep it would be advisable (and it is required by some European concrete codes) to place special longitudinal reinforcement in the middle portion of a beam stem.

## SHEAR AT POINTS OF CONTRAFLEXURE[15,16]

We have seen that there is good reason to assume that concrete and web reinforcement will act together for cases of combined bending and shear.

That can only happen if a portion of the concrete section is uncracked, as is generally the case where there is some bending. This also forms the basis for the ordinary shear stress formula for reinforced concrete. We cannot, however, assume that this formula will apply if one of the basic external load conditions such as bending is absent.

When calculating the stresses in a reinforced concrete section, it is usual to designate the condition before cracking as "Case I." When a crack forms and only a part of the concrete section is effective, the condition is designated as "Case II." The usual shear stress formula for reinforced concrete with its corresponding stress diagram is based on the conditions of Case II. This assumes cracks to be perpendicular to the axis of the member and that there will be an uncracked portion from the neutral axis to the compression side of the beam.

To follow the action of the stressing of the member, it is necessary to start with Case I. We know that concrete has its greatest weakness in tension and when it cracks it will ordinarily start along a line perpendicular to the principal tensile stresses. This cracking line is never vertical at any point on a beam acted on by transverse forces where there is little or no bending moment. The assumption of vertical cracking at such a point to suit our usual shear stress formula is therefore not in order. Furthermore, since the tensile stresses perpendicular to a diagonal plane will extend from top to bottom of the beam and will be of greatest magnitude near midheight, we cannot generally figure on the crack stopping before it has passed through the complete section. There is in such an instance no Case II, but only Case I and a second condition with the concrete section entirely inactive. The whole shear and direct tension will then have to be carried by the steel without any help from the concrete. If the steel is not strong enough for that purpose, the result will be complete failure. Such a condition is encountered at a point of contraflexure of a continuous beam since there is no bending at that point but only transverse shear. There may be, in addition, either direct tension or compression acting in the longitudinal direction of the beam. If the direct load is compression, the diagonal tension component will be smaller, but if there is longitudinal tension, the combined diagonal tension component will be greater with a correspondingly decreased factor of safety.

The allowable shearing stresses found by the ordinary shear stress formula are based on tests of simply supported beams. Even if the stresses found by this formula are not exact, they still form a useful basis for comparison for similar conditions of loading, shape, and support. If conditions are quite dissimilar, as at a point of contraflexure, an approximate formula may lose all its value as a measure of comparison. The originators of the formula may not have been aware of this limitation, or at least have failed to draw it to the attention of others; that the formula gives only relative values has been known for many years (Reference 5, p. 343).

If we want to find what the approximate shearing stresses are at and near a point of contraflexure before any cracking has taken place, it is necessary

to use the formula for homogeneous sections. This formula will give a unit maximum shear stress of $v = 1.5V/bh$ at the neutral plane of the section and the stress will decrease parabolically to zero at top and bottom of the beam. The presence of reinforcing steel may modify the stress to some extent. Investigations by Mörsch have indicated a coefficient of 1.62 instead of 1.5 where the bending moment is low and the concrete uncracked.[17] This small difference, which may in itself be variable, is of little importance. The important point is that the maximum principal tensile stress at points of contraflexure is not at the bottom or at the top of a beam but rather at the centroid of the section. Furthermore, since the weak section for principal stresses is not vertical, but inclined, there will be compressive stresses induced from bending both at top and bottom where the incline reaches the top and bottom surfaces. This again increases the tensile stress at the neutral plane of the section. However, whether a crack will first start at top, bottom, or center will likely depend on local weaknesses in the concrete.

If a through crack forms at a point of contraflexure, neither the shear formula for homogeneous beams nor the usual shear formula for reinforced concrete will apply because there will be no shearing strength of the concrete left for transfer of shear across the crack. In a case like this, the structural stability of the member depends either on the ability of the concrete alone to take the complete diagonal tensile stress or, if the concrete cracks, the complete shear and longitudinal force must be taken by the reinforcement. Putting in web reinforcement for only two-thirds of the shear, for instance, is equivalent to expecting web reinforcement to carry one-third extra stress after it gets into action due to formation of a through crack.

It is true that the positive and negative bending moments will increase relatively fast from a point of contraflexure. However, if we assume the weak plane in diagonal tension to be at 45 deg (and it will be steeper in most cases), the distance available for getting concrete into compression will be, at the most, only half the depth of beam on each side of the contraflexure point. Since, in the majority of cases, there will be initial tension in the concrete due to restrained shrinkage, we cannot safely figure on any portion of concrete being in compression along the potential diagonal crack line. Furthermore, any increase in compression at both top and bottom of beam must create a corresponding additional tension on the center portion so there will be no real help from such compression.

The horizontal inward reaction in a rigid frame will be helpful, particularly in the end spans, to put some horizontal compressive stresses into the beam. Internal columns, if fairly rigid, will decrease this compression. Interior spans will therefore, and because of axial shortening of the beam due to shrinkage and temperature decreases, be in the worst position even if the shearing stresses are the same.

There is little doubt that the most suitable web reinforcement at points of contraflexure are bent-up bars or sloping stirrups. A crack at this position may be as steep as 60 deg and perhaps even steeper due to the presence of

longitudinal forces.[18] In such cases, the shear distance adequately covered by a vertical stirrup may be rather short. Bent-up bars will cover a greater distance and will not be liable to destruction one by one to the same extent as stirrups, which may be stripped off progressively.

There is generally some initial internal stress in a beam caused by shrinkage. For a section containing reinforcement, this will cause compression in the steel and tension in the concrete. Assuming, for instance, a shrinkage coefficient for unreinforced concrete of $m = 0.0003$, $E_s = 29{,}000{,}000$ psi, $p = 1.5$ percent, and $n = 10$.

$$f_c = mE_c \frac{np}{1 + np} = 0.0003 \times \frac{29{,}000{,}000}{10} \times \frac{10 \times 0.015}{1 + 10 \times 0.015} = 113 \text{ psi tension}$$

$$f_s = \frac{f_c}{p} = \frac{113}{0.015} = 7533 \text{ psi compression}$$

The coefficient of shrinkage used here is smaller than may be expected for indoor conditions and completely dried out concrete. On the other hand, there will be a plastic flow of concrete which will reduce the shrinkage deformation somewhat.

The beam length will also decrease in accordance with the deformation caused by shrinkage shortening of the complete system. If the supports are of a kind which will restrain such longitudinal deformation, the result will be a further increase in concrete tension. The maximum tension created by concrete shrinkage would occur where all longitudinal contraction was prevented by support resistance. Complete hindrance of contraction is infrequent, but a considerable resistance is rather common. It is, unfortunately, difficult or impossible in many cases to determine this resistance accurately.

There is still another form for shrinkage stress which might not be negligible. In a member of large cross section, drying out of surplus water must necessarily start from the outside. If this drying process takes place quickly, there will be large shrinkage stresses set up at the exterior part of the member while the interior will be in compression and act in about the same manner as external support resistance. It is quite conceivable that under unfavorable conditions, the external concrete may be overstressed and get thin cracks not easily visible. The complete concrete section may not be at its weakest during this period of drying since the internal concrete is still in compression and thus will decrease the longitudinal shortening of the member. A similar condition of differential shrinkage stresses could occur in an I-shaped member where the thinner web would dry out more quickly than the bulkier flanges.

It is clear that the contribution from shrinkage stresses to principal tensile stresses could be quite large. We might get into a situation where the tensile strength of concrete is overcome even if the nominal shearing stress is not unduly high.

Let us now consider the combined stress condition at a point of contraflexure (Fig 2). We will at first assume that there is no crack and that section $A$-$A$ represents a potential plane of weakness perpendicular to the principal tensile resultant stress. The stress distribution on the portion of the beam to the right would then be about as indicated in Fig 2a. If a crack forms there must then be a redistribution of stresses if equilibrium is to be maintained since the shear component $S$ will be destroyed (assuming no dowel action). This new stress condition is shown in Fig. 2b. It is clear that the tension component $T$ in Fig. 2a is causing the crack and not the shear component $S$, since concrete is relatively strong in shear.

The force diagram shown in Fig. 2b would have to be modified if there were an external horizontal force acting on the beam. If this force was a tension, the direct compressive stress in the horizontal reinforcement would decrease and even turn into tension if the external force became greater than $H_1 + H_2$. With stirrups only taking all shear, there would of course always be tension in the horizontal reinforcement if the beam had an external tensile load.

The question may be asked if it is necessary in all cases to provide web reinforcement at points of contraflexure. The writer does not think that this

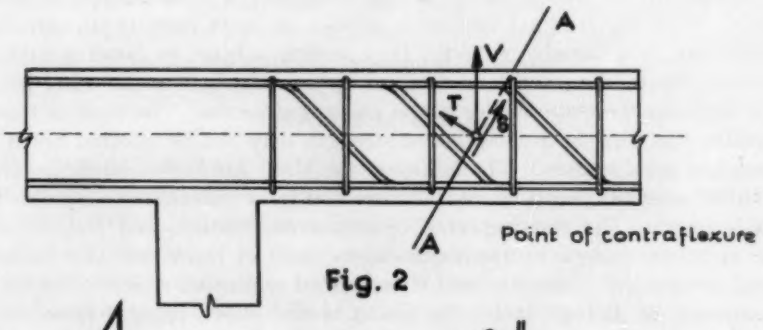



Fig. 2

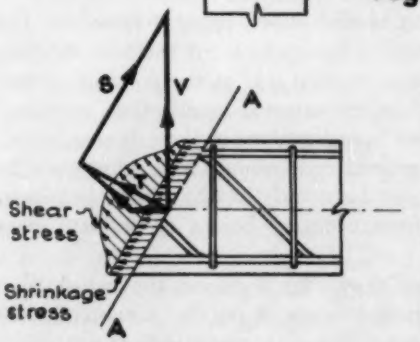



Fig. 2 a

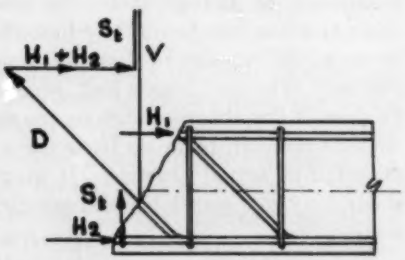



Fig. 2 b

Fig. 2—Stress conditions at a point of contraflexure

is the case. We know that concrete can take some tension and that also applies when such tension acts in a diagonal direction. Continuous reinforced concrete slabs, for instance, have given little or no trouble at points of contraflexure because the diagonal tension in most cases will be low and on the safe side.

It has been mentioned that the presence of longitudinal tension will cause a diagonal crack to be steeper than 45 deg. If there were no shear, the crack would of course be about 90 deg with the longitudinal axis. Conversely, with longitudinal compression, such as for instance in a precompressed concrete member, the diagonal crack should occur at an angle smaller than 45 deg. While sloping web reinforcement would be most advantageous for a member subject to axial tension, vertical stirrups should be better suited for a precompressed concrete member.

Another difficult question to answer is what constitutes the most dangerous section in shear. Is it the section close to an interior support where both shear and bending moment will generally be a maximum, or is it at a point of contraflexure where there may be a high shear but no bending? These questions are not answered completely by tests and experience. It seems probable, however, that longitudinal tension from shrinkage or temperature deformation will make the sections at points of contraflexure the most dangerous in many cases. Such longitudinal tension is present in most large continuous structures, but to a variable extent. In a section subject to bending and axial tension, the tension may tend to open up a crack, but on the other hand, it will decrease the compression in the uncracked section. Because of this it is possible that the shear-compressive strength may not be affected much by a moderate axial tension. The failure of the U. S. Air Force warehouse frames occurred near the points of contraflexure and tests indicated these points to be the weakest. The tests reported by Anderson, Elstner, and Hognestad are the only ones known by the writer where the test beam was also subject to axial tension.[18,19] German tests of restrained reinforced concrete beams were mentioned by Balog[20] in his discussion of the above paper. However, the three German test beams had haunches of a form which cut the unit shearing stress at the support to a small fraction of what it was at the point of contraflexure.[8] The test beams had, in addition, no external longitudinal restraint. In view of the above conditions the shear investigations of these beams, in the writer's opinion, form no basis for a general conclusion about what may be expected in actual practice. It may also be noted that haunched beams in shear may act somewhat differently from ordinary beams of uniform cross section.

We may perhaps assume, until further knowledge is gained, that when some longitudinal restraint is present, shearing stresses at points of contraflexure are more dangerous than shearing stresses of the same magnitude at any other point. It seems probable that shearing stresses of the same magnitude will be more dangerous at a point of contraflexure than near the support where there are also compressive stresses from bending. If this assumption is correct,

we should then cut the allowable shearing stresses at points of contraflexure below what is allowed near an interior support. It is fortunate that in most cases the shear at points of contraflexure in a uniformly loaded member complies with such a requirement. In practice it may vary somewhere between 50 and 70 percent of the shear at the face of support. For a member of uniform cross section, this automatically provides a corresponding reduction in diagonal tension. This may not be true for concentrated loading and for nonuniform cross sections. The writer thinks that if the maximum allowable shearing stress at the face of an internal support does not exceed what is allowable for unreinforced concrete, then no special web reinforcement is needed at points of contraflexure unless conditions are such that the shearing stress at this point exceeds about two-thirds of the allowable shearing stress. If, however, the allowable shearing stress at face of support is exceeded, web reinforcement for at least two-thirds of the shear at support and for the full shear at and somewhat beyond the extreme location of points of contraflexure should be provided. For most cases this rule will allow a uniform spacing of shear reinforcement from the support to the nearest location of point of contraflexure, and that will simplify both design and placing of shear reinforcement. Special precautions should be taken however, when the continuous line of beams is long and heavily restrained against longitudinal movement and where most of the load is of a permanent nature. The same applies to marginal beams and other beams subjected to considerable torsion.

## FLANGE EFFECT

In the 1956 ACI Building Code, beams subject to T-beam action were excluded from the increased shear reinforcement requirements [Section 801(e)]. The reason for this exclusion is not clear. No other reinforced concrete code, to the writer's knowledge, gives any special preference in shear to T-beams. The contrary view is more often held: that T-beams should always have some web reinforcement regardless of shear intensity to tie flange and stem securely together.

It would seem that at a freely supported end with ordinary proportions of flange and stem, the flange will have a negligible effect on preventing initial diagonal tension cracks in the stem. The effect will also be small in resisting widening of the cracks because this will depend mostly on the horizontal and vertical reinforcement. It is reasonable to assume that the flange will have some effect on the ultimate load capacity because of some increased resistance in combined shear and compression. For beams with relatively thin flanges the beneficial effect might be quite small. It is clear that when comparing T-beams with a rectangular beam of the same total depth, the actual lever arm of internal stress resultants must be used.

It is rather difficult to find if a flange of any proportion will be of any benefit at a point of contraflexure. The initial shrinkage stress would certainly be as large in the flange as in the beam itself and possibly larger because of

less reinforcement resisting shrinkage shortening. It is even possible that the adjacent slabs forming the flanges might crack before the stem due to shrinkage stresses if the system is heavily restrained from contraction. Taking the above into consideration, tests of isolated flanged beams might not give results properly applicable to a more complete structural system. Considering also the large variation in flange proportions, the flange effect becomes so uncertain and variable that it seems safest to disregard it and to use the same rule for shear reinforcement as for rectangular beams. In the case of marginal beams subject to torsion, an assumed benefit from T-beam action appears to be especially unjustified.

On the portion of beam with negative bending moment and with a top flange, there can, of course, be no question about T-beam action. The flange must crack if the tension reinforcement gets into action, in which case the beam will act as a rectangular section in both bending and shear.

The writer is aware that test results of two T-beams[14] show some advantage of T-beams over rectangular beams with vertical stirrups (but no advantage where the web reinforcement of the same strength was placed on a slope). These two laboratory tests with isolated T-beams having no special longitudinal restraint certainly do not seem to form a sound basis for excepting T-beams of all forms and any flange to stem proportion from the general requirements of shear reinforcing in beams.

Some European codes require a minimum of web reinforcement along the complete length of both rectangular beams and T-beams. Stirrups are mostly used for that purpose. How effective and necessary stirrups are as shear reinforcement near the center of a beam with nearly vertical tension cracks may be questionable. However, they help to form a rigid skeleton and will act as ties for the top bars in a rectangular section. At the sections with negative bending moments, stirrups will also be useful as ties for the reinforcement in compression. Whether or not compression reinforcement is theoretically required, we may be sure that the influences of shrinkage and creep will generally induce considerable compression into the bottom bars. The writer makes it a rule to use stirrups for the complete length of any beam of ordinary size and importance.

## SHEAR CONDITIONS WHERE BOTH BENDING AND SHEAR ARE MAXIMUM

This condition is encountered near an interior support of a continuous beam uniformly loaded, but may also be found at any other place on beams with concentrated loads. The direction of cracks at such places will be much steeper than 45 deg, and it is not certain that cracks will be crossed by an ordinary vertical stirrup. In spite of this obvious fact, it has so far been customary to design web reinforcement for shear only and to disregard the influence of bending stresses on crack direction. The writer has some doubt if uniform safety is obtained by means of such simplification. Is it not likely that we must rely entirely on the shearing resistance of the uncracked concrete section,

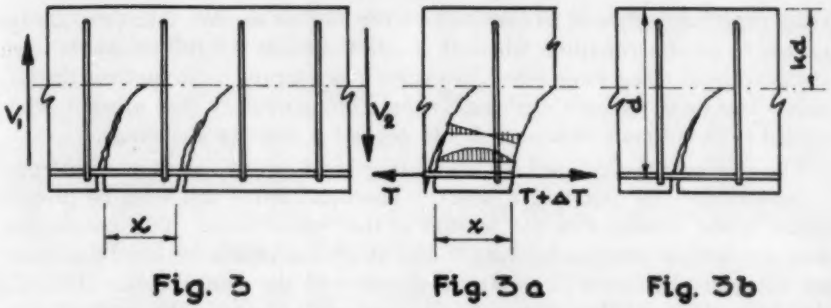



Fig. 3 Fig. 3a Fig. 3b

Fig. 3—Cracked beam with none of the cracks crossed by shear reinforcement

with a little help from dowel action, to carry the complete shear if web reinforcement does not cross a crack?

Fig. 3 shows a portion of a beam where there are two cracks with relatively slight slopes at a distance apart of $x$, none of them crossed by shear reinforcement. The shearing force acting on the cantilever in Fig. 3a is evidently $\Delta T$. The shear will then be $1.5(\Delta T/bx)$ in which $\Delta T$ will depend on the depth of beam and the bending moment while the distance $x$ will be chiefly a function of the bonding resistance.

If we now consider what takes place at one of the cracks, we find an entirely different condition. The tension in the main reinforcing bar will be the same at each face of the crack. No horizontal shear is transferred across the crack by the steel and none by the concrete itself across the crack. No vertical shear can be transferred by the concrete from one side of the crack to the other side, but there may be some shear transferred by dowel action of the main reinforcement. It is likely, however, that most of the shear and all compression from bending must be carried by the remaining uncracked concrete section above the neutral axis and the true principal stresses, where they have to flow past a crack, will be rather complicated.

It is easily seen that the ordinary formula for shearing stresses will not, in this case, give a true picture of stress conditions and its value, even for comparison, may not be too good. More investigations and tests of such problems are needed.

## ANCHORAGE

The general acceptance of the improved type of deformed reinforcing bar has made possible a considerable increase in allowable bond stress. However, the increased bond stress will also increase the tendency of splitting. Allowable bond stresses have been determined on the basis of pull-out tests where the concrete specimens were so large that splitting was not too critical a factor. There can be little doubt that where splitting is prevented, the bond stresses allowed by the codes are safe. However, reinforcing steel is placed in most

cases near the surface of a concrete member and we cannot, therefore, always expect to have a condition where it is safe to utilize the full allowable bond stress without using some other means such as stirrups to strengthen the section. Just as in the case of shear it should be mandatory that a beam, when loaded to its ultimate capacity, should not fail in bond or anchorage.

The ordinary theory used for calculating bond stresses assumes two transverse sections one unit length apart. The bond stress will then be proportional to the difference in bar tension at the two sections. This assumption does not hold in practice because cracks at critical points for bond and shear are not generally formed in a 90 deg direction to the neutral axis. It should also be understood that transverse stirrups can only take the vertical component of the shearing stress. The horizontal component, as has been mentioned earlier, must be taken by the horizontal reinforcing steel and will add to the tension in this steel over and above what is caused by bending alone. This fact has generally been overlooked and little attempt has been made to evaluate the additional tension in the main reinforcement. An analysis of a particular case was published by Rausch, and discussed by other German engineers.[21] Rausch's solution is based on a crack forming at a 45-deg angle and that the complete vertical shear is taken by the stirrups.[6] However, cracks do not always form at 45 deg and American codes allow some part of the shear to be taken by the uncracked concrete. A more general theory, of which the Rausch formula is only a special case, has been worked out and published.[16]

In Fig. 4 assume a crack at an arbitrary angle. $S$ is an arbitrary proportion of shear taken by the stirrups, while the rest is taken by the uncracked concrete in the compression zone at $O$. Let us disregard the weight of the beam end itself, although it would not be difficult to include this in the analysis.

With moment about $O$, we have:

$$R(b + x) - Tz - \frac{Sx}{2} = 0$$

$$\text{or } T = \frac{Rb}{z} + \frac{Rx}{z} - \frac{Sx}{2z}$$

$$\text{from which } T = \frac{M_b}{z} + \frac{Rx}{z} - \frac{Sx}{2z}$$

For $x = 0$, that is, the crack being vertical, the two last terms of the equation disappear and:

$$T = \frac{M_b}{z}$$

which is in accordance with the accepted bending moment theory. There will be no excess bar tension for that case.

For $x = z$ and $S = R$, that is, the crack being at 45 deg and the complete shear taken by the stirrups, we have:

$$T = \frac{M_b}{z} + R - \frac{R}{2} = \frac{M_b}{z} + \frac{R}{2}$$

which is the Rausch formula.

If all the shear is taken by the concrete at $O$, that is, there are no stirrups, and we assume the crack at 45 deg or $x = z$, then:

$$T = \frac{M_b}{z} + R$$

It is seen that without stirrups the excess bar tension is twice that given by the Rausch formula.

If $x$ is greater than $z$, which may well happen if the top of a secondary crack joins the primary crack, and there are no stirrups, it follows that the excess stress may become much greater than the magnitude of shear $R$. The formula can, of course, also be easily solved for any assumed proportion of shear taken by stirrups.

If we have no stirrups, it is evident from simple statics that the tension $T = M_o/z$ will be constant regardless of what the downward slope from point 0 is. If the slope becomes flatter, the difference between the stress in the bar at the crack, and just to the left of the crack, will become increasingly greater.

The apparent increase in bar tension where no stirrups are used, or only a part of the shear taken by stirrups, would indicate that it is desirable to use an ample amount of shear reinforcement.

It is rather disturbing that the above deduction more or less plays havoc with the accepted theory of bond stress as given in the textbooks on reinforced concrete. Instead of a differential stress corresponding to the differ-

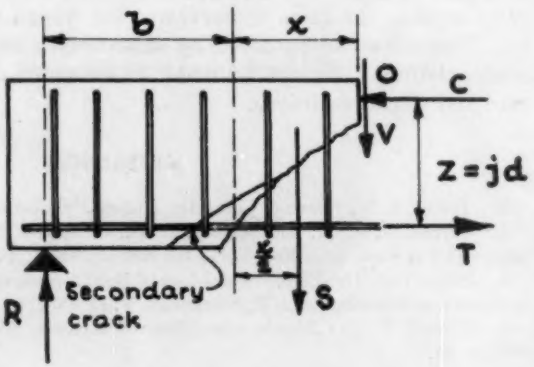



Fig. 4—Anchorage example problem

ence in moment tension at two points a unit length apart, we may have a unit differential stress near a crack corresponding to sections as far apart as the depth of the beam itself and even more.

Another conclusion which may be drawn from the analysis, is that caution should be used not to cut bars too short when stopping them beyond a point of assumed zero bending moment. It is quite possible that the bars may carry much more stress near their terminal ends than has hitherto been assumed.

In the case of a rectangular beam, there should be continuous longitudinal reinforcement on top along the complete length of the beam and splices in such reinforcement should have a sufficient overlap to develop the complete strength of a bar. A rectangular beam should be secured by reinforcement on the compression side against lateral forces even if such forces are not subject to calculation. It is almost as illogical to leave a rectangular beam unreinforced on the compression side as to eliminate reinforcement in a column when the concrete itself does not need any help from steel to carry the load.

When a crack has formed, it is clear that bond stresses near the crack on both sides of it will be much higher than obtained fron the usual formula. This high bond stress together with dowel action will frequently cause formation of a secondary crack which may join the original crack higher up on the beam. This may result in practically a loose piece of concrete being formed. If this should happen, the distance $x$ in Fig. 4 will again increase with a corresponding increase in bar tension.

## CONCLUSION

In this discussion of some of the many problems in shear, the writer is aware that some of his views may be controversial and should receive more investigation. It is hoped that this paper will further the interest in shear problems so that a concerted effort may be made to investigate and solve that which is still unclear. In the meantime, the writer would advise a designer not to skimp on web reinforcement, especially on long strings of continuous beams where considerable resistance to longitudinal contraction should be expected. The more important a structure is and the longer the span or the larger the cross section, the more conservative the design of web reinforcement should be. The money to be saved by skimping on such reinforcement is seldom worth the time required to make an elaborate analysis based on uncertain load and stress conditions.

## REFERENCES

1. Talbot, A. N., "Tests of Concrete: I. Shear," *Bulletin* No. 8, University of Illinois, 1906.
2. Turneaure, F. E., and Maurer, E. R., *Principles of Reinforced Concrete Construction*, John Wiley & Sons, Inc., New York, 4th Edition, 1932, pp. 15-16.
3. Rehm, G., "The Fundamental Law of Bond," *Symposium on Bond and Crack Formation in Reinforced Concrete*, RILEM, Stockholm, V. II, 1957, pp. 491-498.
4. Mörsch, E., *Der Eisenbetonbau*, Konrad Wittwer, Stuttgart, 6th Edition, V. 1, Part 1, 1926, p. 84.

5. Suenson, E., *Jernbeton*, Blumes Boghandel, Copenhagen, 4th Edition, 1931, pp. 338-340.

6. Rausch, E., *Drillung, Schub und Scheren im Stahlbetonbau*, Deutsche Ingenieur Verlag, Düsseldorf, 1953, pp. 66-79.

7. Laupa, A., Siess, C. P., and Newmark, N. M., "Strength in Shear of Reinforced Concrete Beams," *Bulletin* No. 428, Engineering Experiment Station, University of Illinois, Mar. 1955, pp. 7-73.

8. Mörsch, E., *Die Bemessung im Eisenbetonbau*, Konrad Wittwer, Stuttgart, 5th Edition, 1950, Chapter 12.

9. Bay, Herman, "Schubbewehrung und Bruchsicherheit beim Stahlbetonbalken," *Beton- und Stahlbetonbau* (Berlin), V. 52, No. 7, July 1957, pp. 156-162.

10. Moretto, Oreste, "An Investigation of the Strength of Welded Stirrups in Reinforced Concrete Beams," ACI JOURNAL, V. 17, No. 3, Nov. 1945 (*Proceedings* V. 42), pp. 141-162.

11. Moody, K. G., Viest, I. M., Elstner, R. C., and Hognestad, E., "Shear Strength of Reinforced Concrete Beams. Part 1—Tests of Simple Beams," ACI JOURNAL, V. 26, No. 4, Dec. 1954 (*Proceedings* V. 51), p. 318.

12. Saliger, Rudolf, *Der Stahlbetonbau*, Franz Deuticke, Vienna, 7th Edition, 1949, pp. 343-344.

13. Probst, E. H., "Ein Beitrag zue Frage der Schubsicherung von Eisenbetonbalken," *Der Bauingenieur* (Berlin), V. 12/13, 1931.

14. Elstner, R. C., Moody, K. G., Viest, I. M., and Hognestad, E., "Shear Strength of Reinforced Concrete Beams. Part 3—Tests of Restrained Beams with Web Reinforcement," ACI JOURNAL, V. 26, No. 6, Feb. 1955 (*Proceedings* V. 51), pp. 525-539.

15. Rensaa, E. M., "Letter to Editor," *Engineering News-Record*, V. 157, No. 18, Nov. 1, 1956, pp. 10, 14.

16. Rensaa, E. M., "Shear, Diagonal Tension and Bond Stresses in Reinforced Concrete Beams," *The Engineering Journal* (Montreal), V. 40, No. 10, Oct. 1957, pp. 1464-1471.

17. Mörsch, E., *Der Eisenbetonbau*, Konrad Wittwer, Stuttgart, 6th Edition, V. 1, Part 2, 1926.

18. Anderson, Boyd G., "Rigid Frame Failures," ACI JOURNAL, V. 28, No. 7, Jan. 1957 (*Proceedings* V. 53), pp. 625-636.

19. Elstner, Richard C., and Hognestad, Eivind, "Laboratory Investigation of Rigid Frame Failure," ACI JOURNAL, V. 28, No. 7, Jan. 1957 (*Proceedings* V. 53), pp. 637-668.

20. Balog, Louis, discussion of "Rigid Frame Failures," ACI JOURNAL, V. 29, No. 6, Part 2, Dec. 1957 (*Proceedings* V. 53), pp. 1288-1295.

21. Rausch, E., "Zur Shubsicherung im Stahlbetonbau," *Der Bau* (Düsseldorf), No. 1 and 2, Jan. 1949, pp. 9-12, 34-36. Discussion by E. Mörsch, No. 22, Nov. 1949, pp. 538-539; by E. Rausch, No. 5, Mar. 1950, pp. 108-110.

22. Richart, F. E., and Larsen, L. J., "An Investigation of Web Stresses in Reinforced Concrete Beams. Part II—Restrained Beams," *Bulletin* No. 175, University of Illinois, 1928.

**Discussion of this paper should reach ACI headquarters in triplicate by Mar. 1, 1959, for publication in the June 1959 JOURNAL.**

Title No. 55-46

# STRAINS

## in beams having diagonal cracks

By D. Watstein and R. G. Mathey

SEVERAL REINFORCED CONCRETE BEAMS were tested to determine the validity of the usual assumptions that longitudinal reinforcement does not transfer vertical shear across a diagonal tension crack and that the maximum compressive strain within the shear span is developed at the extreme fiber. Extensive strain measurements in the steel and the concrete indicated that a plane section did not remain plane following development of a diagonal crack and that the maximum compressive strains in the concrete occurred some distance below the extreme fiber. The longitudinal reinforcement was found to carry considerable vertical shear across a crack, but this force decreased rapidly as the load approached the maximum.

IN ANALYSES OF DATA on shear strengths of reinforced concrete beams it is commonly assumed that tension reinforcement does not transfer vertical shear across a diagonal tension crack and that the maximum strain in the concrete at the critical section of the shear span occurs at the outermost fiber. It is also usually assumed that the maximum value of this strain in the shear span of a beam failing in shear is of the same order of magnitude as the maximum strain in a flexural failure.

Several reinforced concrete beams designed to fail in shear were tested at the National Bureau of Standards in order to investigate the validity of these assumptions. The instrumentation used in this investigation was designed to determine the distribution of strain within the shear span and the magnitude of the vertical shear carried by the longitudinal reinforcement across a diagonal tension crack.

### TEST PROGRAM AND SPECIMENS

The nine beam specimens used in this study were designed and instrumented to yield information on the distribution of strain in both concrete and steel during the progressive formation of diagonal tension cracks and to furnish the means of computing the shear carried by the longitudinal rein-

Received by the Institute Mar. 24, 1958. Title No. 55-46 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55). Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1959. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

ACI Members D. WATSTEIN and R. G. MATHEY are colleagues at the National Bureau of Standards, Washington, D. C., where the research reported in their paper was conducted. ● Mr. Watstein, who is chief of the Structural Engineering Section of the Bureau, will be remembered for several earlier JOURNAL papers describing research on control of width of cracks and distribution of bond stresses in reinforced concrete. ● Mr. Mathey, structural research engineer in the Bureau's Structural Engineering Section, now works primarily on studies of reinforced concrete beams containing high yield strength reinforcement.

forcement across these cracks. In addition to extensive strain measurements in both the steel and concrete, it was also necessary to plot the patterns of the diagonal tension cracks to compute the vertical shear transferred by the longitudinal reinforcement.

The beam specimens are described in Table 1. Two types of beams were used in this investigation. One of these was Beam A-15 illustrated in the top of Fig. 1. Beam A-15, whose cross section was 6 x 15 in., had a 9-ft span and was loaded at quarter points. The large number of strain gages placed on the concrete in the shear span of this beam permitted the determination of strains both above and below the major diagonal tension cracks. The second beam typical of those representing Beams B-18 through E-18 is shown in the bottom of Fig. 1. These beams had a cross section 8 x 18 in., a span of 6 ft, and they were loaded at third points. The reactions and applied loads were distributed through steel bearing plates 3½ in. wide and 1 in. thick, which extended over the entire width of the beams.

**TABLE 1—DESCRIPTION OF TEST BEAMS AND RESULTS OF STRENGTH TESTS**

| Beam designation | Longitudinal reinforcement: Number and size of bars | Longitudinal reinforcement: Ratio of reinforcement, percent | Longitudinal reinforcement: Yield strength, psi | Compressive strength of concrete $f_c'$, psi | Modulus* of elasticity of concrete $E_c$, psi | Ultimate load, lb | Ultimate bending moment, in.-lb | Computed† maximum steel stress, psi | Mode of failure |
|---|---|---|---|---|---|---|---|---|---|
| A-15 | 2 #7 | 1.53 | — | 3450 | — | 60,000 | 810,000 | 58,600 | Yield of reinforcement accompanied by compressive failure of concrete |
| B-18-1 | 3 #10 | 3.05 | 38,700 | 3680 | $3.9 \times 10^6$ | 140,000 | 1,680,000 | 33,000 | Shear failure |
| B-18-2 | 3 #10 | 3.05 | 38,700 | 3330 | $4.2 \times 10^6$ | 139,000 | 1,668,000 | 32,600 | Shear failure |
| C-18-1 | 3 #8 | 1.85 | 71,000 | 3710 | $4.5 \times 10^6$ | 130,000 | 1,560,000 | 48,400 | Shear failure |
| C-18-2 | 3 #8 | 1.88 | 67,500 | 3830 | $4.4 \times 10^6$ | 140,000 | 1,680,000 | 51,600 | Shear failure |
| D-18-1 | 2 #6<br>1 #7 | 1.17 | 104,300<br>106,400 | 3720 | $3.4 \times 10^6$ | 120,000 | 1,440,000 | 70,500 | Shear failure |
| D-18-2 | 2 #6<br>1 #7 | 1.16 | 95,500<br>99,000 | 3910 | $3.8 \times 10^6$ | 120,000 | 1,440,000 | 70,300 | Shear failure |
| E-18-1 | 3 #5 | 0.75 | 99,500 | 3250 | $3.9 \times 10^6$ | 99,300 | 1,190,000 | 88,200 | Shear failure |
| E-18-2 | 3 #5 | 0.75 | 99,500 | 3870 | $4.6 \times 10^6$ | 100,000 | 1,200,000 | 88,300 | Shear failure |

*$E_c$ is the value given by slope of secant drawn through point corresponding to stress $\frac{1}{2} f_c'$.
†The steel stresses were computed by formulae based on the straight line theory.
NOTE: Beam A-15 was 6 x 15 in. in cross section, had a span of 9 ft, and was loaded at quarter points. Beams B-18 through E-18 were 8 x 18 in. in cross section, had a span of 6 ft, and were loaded at third points.

All gages placed on the concrete and the reinforcing bars were bonded wire resistance strain gages. Gages were placed on both sides of the concrete beams in order to obtain the average values of strain in the concrete. Bonded wire gages, suitably waterproofed, were also placed in pairs on the reinforcing bars.

Note in Fig. 1 that Beam A-15 had an overhang of 12 in. and the other beams had an overhang of 18 in. This long overhang was provided instead of the usual short overhang needed for suitable anchorage since it was observed in previous tests that beams with short overhang gave more erratic results.

**Materials**

Beam A-15 was made with Type III cement and was tested at the age of 8 days. All the other beams were made with Type I cement and were tested at 28 days. The proportions of the concrete in Beam A-15 were 1:3.2:4.4,

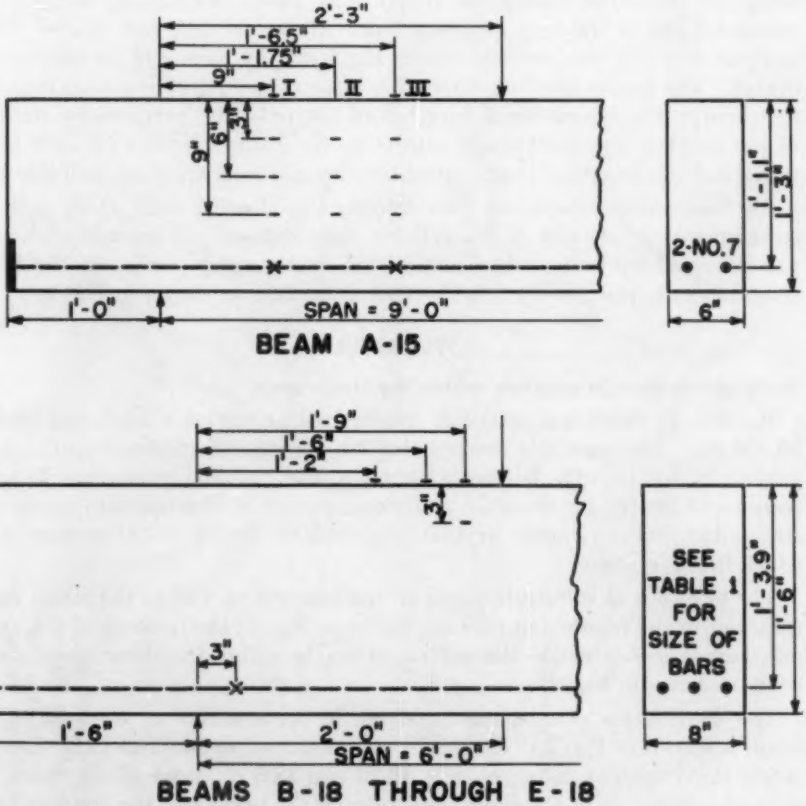



Fig. 1—Details of beams, testing arrangement, and location of strain gages. Gages on concrete and steel are represented by dashes and crosses, respectively

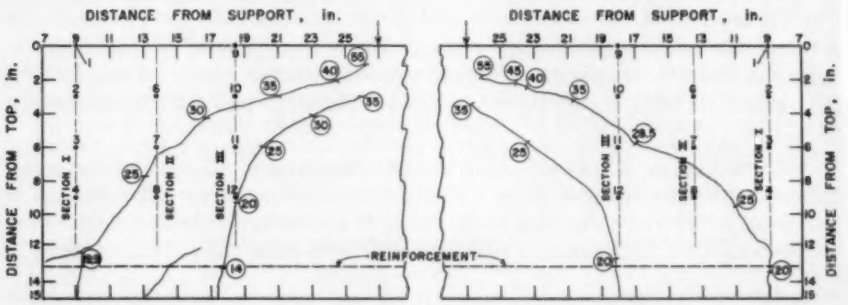



**Fig. 2—Location of gages (by number) on opposite faces of Beam A-15 in relation to principal diagonal tension cracks. Circled numbers indicate load in thousands of pounds at which the extent of the cracks was observed**

while for the other beams the proportions were 1:3.2:4.1, by weight; the maximum size of the aggregate was 1 in. All beams were cast in steel forms and the concrete was consolidated in the forms by means of an internal vibrator. The beams were moist cured in a fog room and were moved from the fog room to the laboratory 2 days before testing. The compressive strength of the concrete was determined with three 6 x 12-in. cylinders for each beam tested and the results of the compressive strength tests are given in Table 1.

All beams were reinforced with deformed reinforcing bars which met the requirements of ASTM A 305-53T for deformations. Adequate anchorage was provided by both embedment in the overhanging portion of the beam and anchorage plates welded to the end of the bars as shown in Fig. 1.

## TEST RESULTS

### Strain distribution in concrete within the shear span

Beam A-15 which was loaded at quarter points carried a maximum load of 60,000 lb. Although this beam failed by yielding of reinforcement accompanied by compressive failure of concrete, the diagonal tension cracks progressed sufficiently far into the compression zone of the beam to produce a strain distribution similar to that observed in Beams B-18 through E-18 which failed in shear.

The locations of the strain gages on the concrete as well as the strain gages attached to the reinforcing bars are shown in Fig. 1; the location of the gages on concrete in relation to the pattern of cracks within the shear span of the beam is shown in Fig. 2.

The distribution of compressive strain in the concrete at one end of the beam is shown in Fig. 3. The strain was measured as Sections I, II, and III of the shear span at distances of 9, 13.75 and 18.5 in., respectively, from the support. Gages were placed on both sides of the beam and the strains shown in all the graphs are the average values. At Section I the strain measured at the compressive face ($y = 0$) increased with load up to about 20,000 lb, at

Fig. 3 (right)—Strain in concrete versus load at Sections I, II, and III (9, 13.75, and 18.5 in. from support, respectively), Beam A-15


DISTANCE BELOW TOP OF BEAM, in.
SECTION I
LEVEL INDICATING POINT OF INTERSECTION OF UPPER DIAGONAL TENSION CRACK AND SECTION III
STRAIN IN CONCRETE

Fig. 4 (left)—Distribution of strain in concrete at Sections I and III (9 and 18.5 in. from support, respectively), Beam A-15. Numbers on curves indicate applied load in thousands of pounds

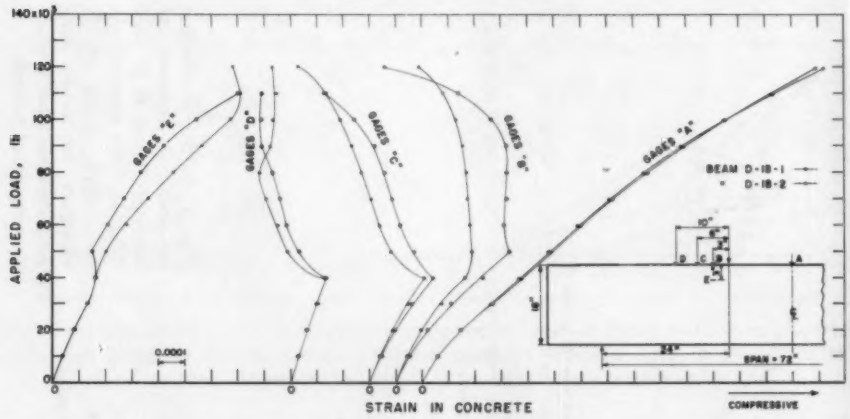



**Fig. 5—Variation of strain in concrete with load in Beams D-18-1 and D-18-2**

which load the principal diagonal tension cracks extended well above the level of reinforcement. From this point on, the compressive strain at the compressive face began to decrease and finally became a small tensile strain. The strain at the compressive face of Section III also followed this pattern. However, the strain ceased to increase with the load at about 30,000 lb but the strain remained compressive and approached zero as the load approached the maximum. The strain in the compression face at Section II was not observed.

At all three sections, the maximum compressive strain determined was at points some distance below the compressive face. At Sections I and II, this distance was 6 in. and at Section III the maximum strain determined was at a distance of 3 in. below the compressive face of the beam or about 1.5 in. above the nearest diagonal tension crack. It is noted that at Section I the compressive strains were greater at distances of 6 and 9 in. from the top of the beam than at the compressive face and at a distance 3 in. below it during the greater part of the load range.

Two of the curves for Section III in Fig. 3 are given for strains at points situated below the upper diagonal tension crack. The strain at these two points indicated that the portion of the concrete between the two major diagonal cracks resisted compression up to the maximum load at which measurements were made.

The distribution of compressive strain at Sections I and III is shown in Fig. 4. The strain distribution was essentially linear up to a load of about 20,000 lb. The diagonal tension cracks extended well above the midplane of the beam in the load range of 20,000 to 25,000 lb. The redistribution of compressive strain occurring at Sections I and III within this load range coincided with the sharply increased steel strain observed at Section I. After the diagonal tension cracks extended well up into the compressive zone of the beam at loads of about 35,000 to 40,000 lb, the location of the maximum

value of compressive strain shifted downward. At Section I the peak value of compressive strain occurred at about the midplane of the beam, while at Section III the maximum strain occurred at a point 3 in. below the compressive face and about 1.5 in. above the diagonal tension crack. The data in Fig. 4 show that concrete below the principal diagonal tension crack resisted compression up to the maximum load at which observations were made.

The variation of strain in the concrete with load is shown for two beams of the second type (Beams D-18-1 and D-18-2) in Fig. 5. The variation of strain in the concrete with load shown in Fig. 5 was typical of all beams in which concrete strain was measured (Beams B-18 through E-18). These beams contained varying amounts of reinforcement ranging from 0.75 to 3.05 percent and were tested in duplicate. All the beams of this type failed by shear at steel stresses ranging from 32,000 to 88,000 psi. The maximum steel stresses computed by the straight line theory are given in Table 1.

The location of the strain gages on the concrete is shown in Fig. 5 for easy reference. It will be noted that the strains recorded by Gages A and B are nearly the same until a diagonal tension crack forms. After the formation of a crack, the strain determined with Gage B continued to increase at a re-

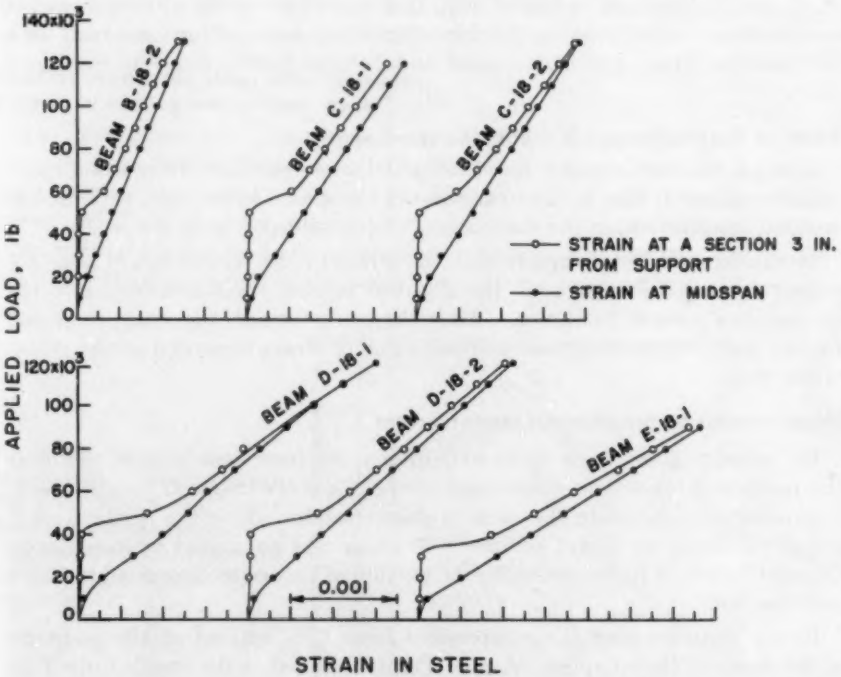



Fig. 6—Strain in steel at a section 3 in. from the support and at midspan in Beams B-18 through E-18

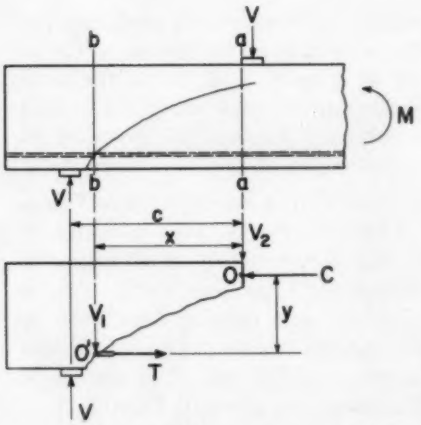



Fig. 7—Free body diagram of a beam following formation of a diagonal tension crack

duced rate or remained substantially constant following a sharp break in the load-strain curve. These alternate actions were displayed by the performance of Beams B-18-1 and B-18-2. In the case of beams D-18-1 and D-18-2 the strain at B decreased rapidly following a load range at which the strain was fairly constant; in one of these beams the strain approached zero while in the companion beam the strain indicated a slight tension. Gages E which were 3 in. below Gages B exhibited consistent behavior. The compressive strain at E increased with applied load up to its maximum except for a slight irregularity at the load at which diagonal tension cracks extended into the compressive zone and approached Gage B. The strain gages at C and D locations indicated with one exception either a fairly constant compressive strain following the formation of a diagonal tension crack, or a compressive strain which decreased and became tensile near the maximum load.

#### Strain in the reinforcement within the shear span

Bonded wire strain gages were placed at various sections of the beam specimens as shown in Fig. 1. The variation of the strains in the steel with load at selected sections within the shear spans and at midspan is shown in Fig. 6.

It can be seen in this graph that the strain in the reinforcement near the support was quite small until the diagonal tension crack extended into the compressive zone of the beam. After the crack formed, the strain increased rapidly and in general approached the value of strain measured at the center of the span.

#### Shear transfer by longitudinal reinforcement

By measuring the stress in the longitudinal reinforcement at a section near the point of intersection of the reinforcement and the diagonal tension crack, it is possible to estimate the vertical shear transferred by the reinforcement across the crack by dowel action. The shear was computed by considering the equilibrium of forces acting on the portion of the beam illustrated in Fig. 7 as a free body.

It was assumed that the compressive force $C$ is applied at the midpoint of the depth of the compressive zone (Point $O$) and that the tensile force $T$ at $O'$ is that determined with the strain gages which may be some distance from $O'$. The assumption regarding the value of $T$ is tantamount to an assumption

that the bond stress between the gage and the crack is negligible. Taking moments about $O$ yields

$$V_1 = \frac{V\,c - T\,y}{x}$$

where the values of $c$, $x$, and $y$ are determined from the plot of the crack pattern.

It is obvious that in order for the free body diagram to be valid, the crack must cross Section *a-a*, Fig. 7. Section *a-a* was chosen as shown in order to minimize the uncertainty in the assumed position of the compressive force $C$.

The relationship between the shear transferred by dowel action and the total shear in the beam is illustrated in Fig. 8 for all beams in this study for which the crack pattern and data on strain in steel were available. In all cases the transferred shear developed its maximum value at the load for which the diagonal tension crack first approached the section at the applied load. Beyond this load, the transferred shear decreased rapidly. In general, the initial value of transferred shear was greater for beams containing larger diameter bars and larger percentages of reinforcement. The ratio of the maximum transferred shear to the total shear in the beams ranged from 0.38 to 0.74 at loads ranging from 0.42 to 0.46 of the maximum.

### Bending within the shear span following formation of diagonal tension crack

The measurement of strain distribution in the "compressive" zone of the concrete indicated that following formation of a diagonal tension crack the beam no longer acts as a structural member in which a plane section remains plane. The fact that the strain at the compressive face of the beam diminished with increasing load and at times even became a tensile strain suggested that the compressive face of the beam must reverse its curvature and become

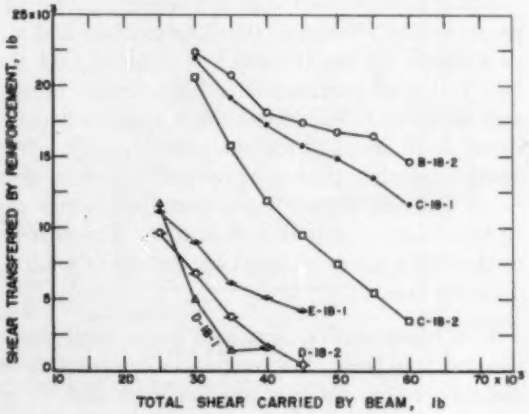



Fig. 8—Shear transferred by longitudinal reinforcement. The shear transferred by the tensile reinforcement across a diagonal crack in a beam was estimated by means of the equation

$$V_1 = \frac{V\,c - T\,y}{x}$$

derived from the conditions of equilibrium in Fig. 7

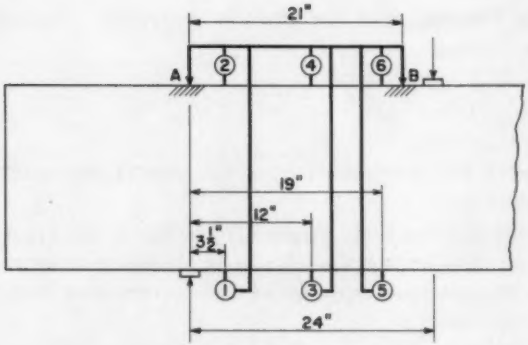



Fig. 9—Schematic drawing showing measurement of displacements of the tensile and compressive faces of beam within the shear span with respect to frame of reference supported at Sections A and B. Numbered circles indicate micrometer dial gages

concave toward the midplane of the beam during the stage of the test corresponding to propagation of the diagonal tension cracks into the compressive zone. In order to check this hypothesis, measurements were made to determine the displacements of the tensile and compressive faces of the beam with respect to a plane of reference indicated in Fig. 9. The results of the measurements are shown in Fig. 10 for two typical beams. It is obvious that the portion of the beam above the diagonal tension crack becomes subject to bending in a manner which causes it to have a curved surface concave toward the midplane of the beam. This occurs abruptly at the load which causes the diagonal tension crack to extend up into the compressive zone of the concrete. Comparison of the load-strain curves in Fig. 6 and the load-displacement curves in Fig. 10 shows that the abrupt change in the profiles of the compressive and tensile faces of the beams corresponds to the sharp increase in the stress in the reinforcing bars at the section 3 in. from the support.

## SUMMARY OF RESULTS

The results showed that the beams containing no web reinforcement undergo an abrupt change in the deformations and stresses following the formation of a diagonal tension crack. It is noted that the test specimens in this study had ratios of longitudinal reinforcement ranging from 0.75 to 3.05 percent and that the ratios of the shear span to depth of beam were small (2.06 for Beam A-15 and 1.51 for the other beams). While this ratio of shear span to depth is smaller than that normally used in shear tests, it was selected in the belief that the phenomenon of redistribution of stress and deformation could be thus more clearly demonstrated. The following phenomena were observed in the shear spans of beam specimens at loads exceeding those causing major diagonal cracks:

1. A plane section remained plane until the formation of a diagonal crack. Beyond this point, the maximum compressive strain occurred at points some distance below the compressive face and, in general, the strain at the ex-

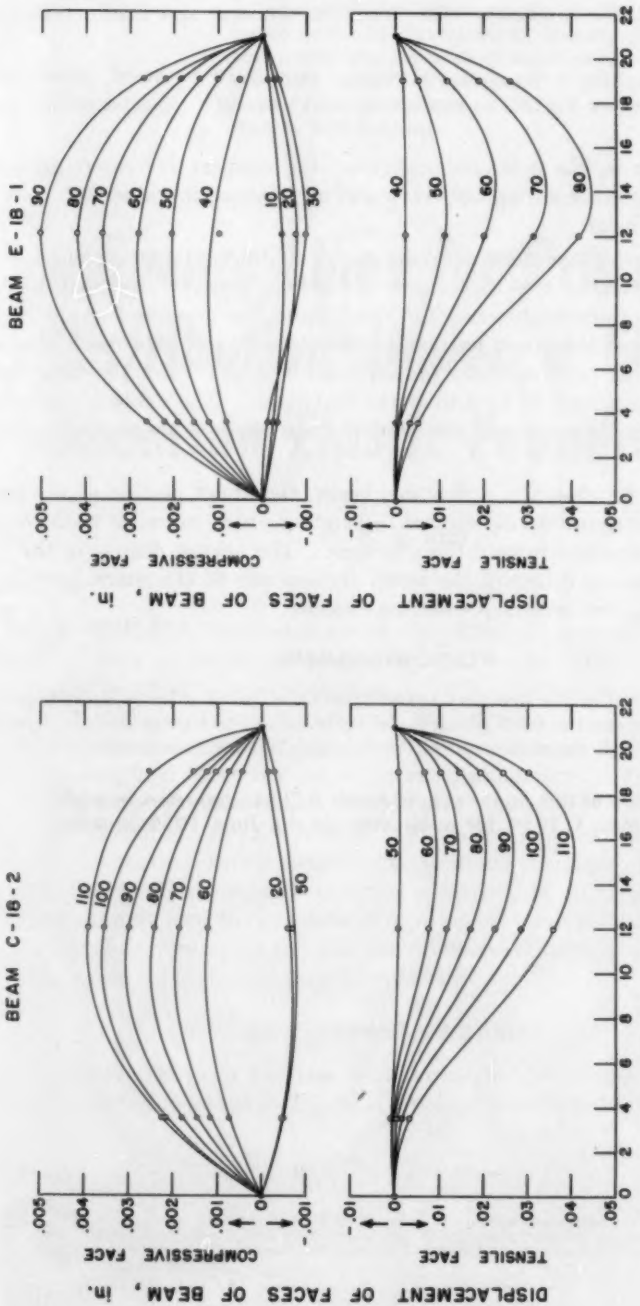



Fig. 10—Displacement of compressive and tensile faces of beam within the shear span measured with respect to frame of reference shown in Fig. 9. Numbers on curves indicate applied load in thousands of pounds. Arrows pointing upward for the compressive face and downward for the tensile face indicate displacements away from the original midplane of the beam

treme fiber of the compressive zone began to decrease and finally became a small tensile strain.

The concrete below the diagonal cracks continued to resist appreciable compressive stresses up to the maximum load at which measurements were made.

2. The strain in the reinforcement near the support increased abruptly following formation of a diagonal crack and approached in value the strain at midspan of the beam.

3. Following the complete development of a diagonal tension crack it is possible to determine the magnitude of vertical shear transferred by the longitudinal reinforcement across the crack from the measured strain in the reinforcement and the crack pattern in the shear span. The vertical shear transferred by the reinforcement ranged from 0.38 to 0.74 of the total shear at loads ranging from 0.42 to 0.46 of the maximum. The value of the transferred shear decreased continuously with load as the crack extended past the load point.

4. Following formation of a diagonal crack, the upper portion of the beam within the shear span became subject to bending which caused it to develop a curved surface concave toward the midplane. The abrupt change in the profile of the compressive face of the beam corresponds to the sharp increase in the strain in the reinforcement near the support.

## ACKNOWLEDGMENT

The work described in this paper represents a portion of the program of research on reinforced concrete supported at the National Bureau of Standards by a grant made by the Committee on Reinforced Concrete Research of the American Iron and Steel Institute.

**Discussion of this paper should reach ACI headquarters in triplicate by Mar. 1, 1959, for publication in the June 1959 JOURNAL.**

**Title No. 55-47 should be of great value in the design of concrete strip footings. A set of influence lines is presented for pressure distribution under a finite beam on elastic foundation. Values of the dimensionless parameter have been selected to cover most of the practical cases. Experiment indicates that influence lines for a parameter of 0.0001 may be used for infinitely stiff footings.**

# Influence Lines for Pressure Distribution Under a Finite Beam on Elastic Foundation

*By K. C. Ray*

THIS PROBLEM has been solved by different authorities in different ways; e.g., Heteyni attempted to solve it analytically. But his solution is not readily usable by structural designers, since it involves working out a long expression with hyperbolic functions. His solution is for a single concentrated load which is seldom encountered in practice on a long beam. Although it can be extended to several loads the calculations involved makes the solution absolutely discouraging to a practical designer. However, Baker's approach to solution of such a beam is more rational with respect to design offices. His method of successive considerations is unique, but the last lap of assumption where subgrade modulus is introduced to calculate deflections of the beam may lead to considerable inconsistencies in results. In this paper an attempt has been made to solve the problem completely and present a set of ready made tables for drawing office use.

## DEVELOPMENT OF TABLES

Fundamental formula for flexure of the beam $AB$ resting on elastic foundation subjected to a load $P$ (Fig. 1) is given by the differential equation

$$EI \frac{d^4w}{dx^4} = q\,b - Kbw \qquad (1)$$

Received by the Institute Mar. 25, 1958. Title No. 55-47 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 55). Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Mar. 1, 1959. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

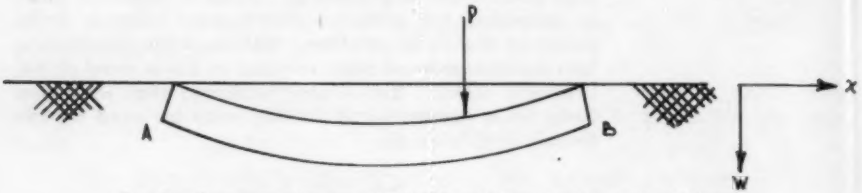



Fig. 1—Beam resting on elastic foundation under action of load

where

$E$ = Young's modulus of the material of the beam
$I$ = moment of inertia of the beam
$q$ = load intensity on the beam acting downward
$K$ = subgrade modulus, expressed as intensity per linear unit of deflection of soil or beam (beam remains in contact with soil)
$w$ = deflection of the beam
$b$ = breadth of the beam

Value of $q$ in above expression is zero at all points excepting the position where $P$ acts as a concentrated load.

The expression $Kw$ represents a state of dense liquid subgrade which is rational and has been accepted by most authorities; Westergaard* also accepted this in his notable paper on pavement design.

**Institute member K. C. RAY is now an executive engineer with the Works and Buildings Department of the State of West Bengal, India. in Calcutta. Designer of several concrete bridges in India, his special interest is in the problem of plates. Last year he received the Indian Road Congress Medal for the best paper of the year for a study of influence fields of reinforced slabs of various spans with two free edges.**

The classical solution of Eq. (1) for $w$ which leads to the solution of $M_x$ and $S_x$ has been obtained by Heteyni† but a numerical solution based on the method of finite difference is suggested in this paper.

In order to find out influence lines for deflection (which is directly proportional to subgrade pressure $Kw$) moments, and shears, some convenient position must be selected for loading. This involves placing the load successively over the positions and working out the values of moments, shears, and deflections at all points for each position of load. The beam is divided into 20 parts, hence $20 \times 20 \times 3 = 1200$ values for above expressions need to be calculated (3 is taken as representing bending moment, shear, and deflection). If however it is decided to work out ten standard sizes of beam, the above number would mount up to $1200 \times 10 = 12{,}000$ times. The enormity of the problem can well be realized by simply looking at the expressions.

*Westergaard, H. M., "Stresses in Concrete Pavements Computed by Theoretical Analysis," *Public Roads*, V. 7, No. 2, 1926, pp. 25-35.
†Heteyni, M., *Beams on Elastic Foundation*, University of Michigan Press, Ann Arbor, 1946, pp. 54-55.

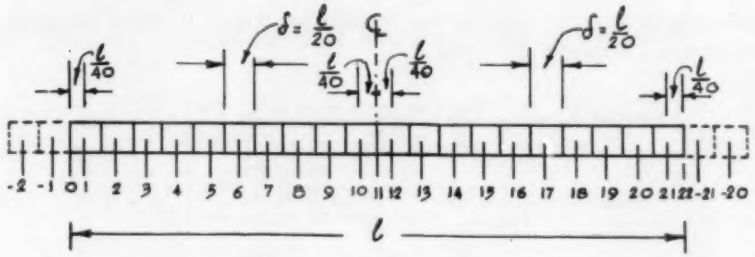



Fig. 2—Beam divided into 20 equal parts

On the other hand however, if a numerical solution is achieved, this will lead to much quicker results, but with a slight degree of approximation. This approximation would be negligible in view of all the uncertainties of the problem and assumptions. Considering above facts, this numerical solution of the problem has been attempted and presented here.

Referring back to the differential equation (1) for flexure, difference equation corresponding to above differential equation is

$$\frac{1}{\delta^4}\left(w_{n+2} - 4\,w_{n+1} + 6\,w_n - 4\,w_{n-1} + w_{n-2}\right) + \frac{Kb}{EI}\,w_n = \frac{q_n\,b}{EI}$$

or

$$w_{n+2} - 4\,w_{n+1} + w_n\left(6 + \frac{Kb\,\delta^4}{EI}\right) - 4\,w_{n-1} + w_{n-2} = \frac{q_n\,b\,\delta^4}{EI}$$

$$= \frac{S_n\,\delta^4}{EI}$$

$$= \frac{P\,\delta^3}{EI} \quad \ldots\ldots\ldots (2)$$

where $S_n$ = load per unit length of the beam at the point considered.

A point load $P$ may be assumed to distribute over the finite interval $\delta$, which means $S_n = P/\delta$, or in the final form the equation can be written as above.

Eq. (2) can now be applied to a finite beam having a practical value for the interval $\delta$. Accordingly the beam has been divided into 20 equal parts as shown in Fig. 2 and Eq. (2) has been applied to satisfy all the sections. It may be observed that for the end sections (Sections 1 and 22), as per difference equations it was necessary to solve for deflections for two imaginary intervals (e.g. $-1$ and $-2$ and $-20$ and $-21$). This introduced 24 unknowns in the problem for a single load position. Applying the equations over 20 divisions, 20 simultaneous equations were obtained, insufficient to solve for 24 unknowns. However, four other equations were obtained from the two boundaries, where bending moment and shear forces are zero.

The difference equations for the boundaries where bending moment and shear force are zero are expressed as

$$\text{Bending moment} = \frac{w_1 - w_2 - w_3 + w_4}{2\,\delta^2} = EI \left.\frac{d^2 w}{dx^2}\right|_{1,-1} = 0 \quad \ldots\ldots\ldots (3)$$

$$\text{Shear force} = \frac{w_1 - 3\,w_2 + 3\,w_3 - w_4}{\delta^3} = EI \left.\frac{d^3 w}{d_x{}^3}\right|_{1,-1} = 0 \quad \ldots\ldots\ldots (4)$$

Thus Eq. (2), (3), and (4) offer a complete solution of the problem. But the coefficient of $w_x$ contains a term $Kb\delta^4/EI$ which describes the entire property of foundation and the beam. If however values for this can be selected to cover the practical ranges, the solution may form a standard for the practical designer. This has been done. Actually cases with values for $Kb\delta^4/EI$ such as 0.001, 0.01, 0.1, 0.2, 0.4 were worked out in detail, and the rest shown in the tables were arrived at by linear interpolation with subsequent graphical corrections.

## INFLUENCE LINE TABLES

Tables 1-11 indicate the influence line for deflections of a finite beam on elastic foundation. The parameter $Kb\delta^4/EI = m$ is dimensionless. The pressure influence line ordinates are found by multiplying the deflection ordinates by $K$, the subgrade modulus. Confusion may arise for values of negative deflection, which indicate that the beam is lifted from the subgrade. This will actually occur in case of heavy loading at scattered individual points on a long beam. In such a case the effective length while calculating the parameter $m$ should be readjusted to the length in contact to solve the problem afresh. However in actual practice such a situation is seldom encountered. An example below illustrates the use of the tables.

Influence line tables for shear force and bending moment have also been worked out, and were published in the *Journal of the Institute of Engineers* (India), V. 38, No. 2, Part I, October 1957.*

### Example

A reinforced concrete foundation beam of dimensions shown in Fig. 3 is subjected to the loading indicated therein. Find the pressure distribution under the beam, also bending moment and shear diagrams. Neglect the dead weight of the beam. $K = 100$ lb per cu in. and $E = 3 \times 10^6$ psi.

To determine the value of the parameter $m$, full concrete section without reinforcement is considered only. However the section with a selected percentage of steel may also be calculated. For simplicity of calculation the former consideration is made here.

$$y = \frac{4^2 \times 1 + 1^2\,(2.5 - 1.0)}{2\,(2.5 \times 1 + 1 \times 3)} = 1.6 \text{ ft}$$

*Text continued on p. 739*

*Copies of these tables, 11 for bending moment and 11 for shear, are available in complete sets only at $4.50 per set to cover the cost of reproduction. Address orders to ACI Publications Department, P. O. Box 4754, Redford Station, Detroit 19, Mich.

## TABULATED VALUES FOR INFLUENCE LINES FOR PRESSURE

$I$ = moment of inertia of beam
$E$ = Young's modulus of material of beam
$K$ = subgrade modulus
$b$ = breadth of beam in contact with soil
$L$ = length of beam

$$m = \frac{Kb}{EI}\left(\frac{L}{20}\right)^4$$

Pressure = ordinate $\times$ $K$ $\times$ load $\times$ $(L/20)^3$ $\times$ $1/EI$

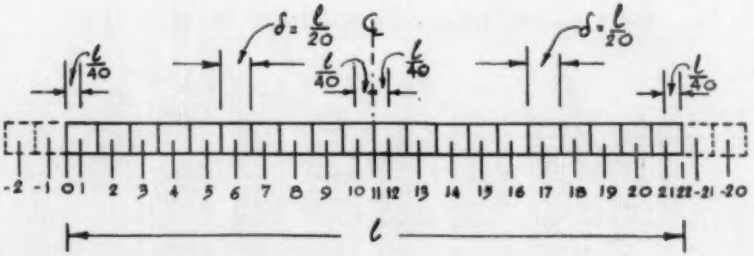



TABLE 1—INFLUENCE LINES FOR PRESSURE, SECTION 1

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 271.0448 | 245.2981 | 142.3112 | 40.1948 | 23.3918 | 6.5877 | 3.7812 | 2.9707 | 2.1613 | 1.3526 |
| 1 | 255.4701 | 231.0763 | 133.5014 | 36.1964 | 20.8132 | 5.5307 | 3.0902 | 2.3510 | 1.7100 | 1.0112 |
| 2 | 224.5258 | 202.6268 | 115.1212 | 28.1995 | 15.7291 | 3.4167 | 1.7082 | 1.2643 | 0.8212 | 0.3824 |
| 3 | 194.3044 | 174.9464 | 97.5143 | 20.8407 | 11.0123 | 1.7495 | 0.7082 | 0.4783 | 0.2484 | 0.0183 |
| 4 | 165.5029 | 148.6029 | 81.0243 | 14.4759 | 7.5210 | 0.6346 | 0.1305 | 0.0556 | −0.0209 | −0.0976 |
| 5 | 138.4553 | 123.9869 | 66.1133 | 9.2529 | 4.6265 | 0.0021 | −0.1262 | −0.1122 | −0.0986 | −0.3081 |
| 6 | 113.3742 | 101.3422 | 53.2121 | 5.1745 | 2.2213 | −0.2810 | −0.1894 | −0.1384 | −0.0880 | −0.3142 |
| 7 | 90.6037 | 80.7877 | 41.5224 | 2.1510 | 0.9015 | −0.3481 | −0.1612 | −0.1073 | −0.0534 | −0.2141 |
| 8 | 70.1516 | 62.3416 | 31.1005 | 0.0412 | −0.1317 | −0.3045 | −0.1058 | −0.0647 | −0.0235 | −0.1117 |
| 9 | 51.7609 | 45.9409 | 22.6614 | −1.3179 | −0.7624 | −0.2207 | −0.0555 | −0.0305 | −0.0057 | −0.0409 |
| 10 | 36.1637 | 31.4603 | 14.4137 | −2.0897 | −1.1131 | −0.1365 | −0.0209 | −0.0115 | 0.0019 | 0.0153 |
| 11 | 28.8285 | 25.1175 | 10.2725 | −2.2569 | −1.1800 | −0.1032 | −0.0114 | −0.0043 | 0.0028 | 0.0099 |
| 12 | 21.4448 | 18.7748 | 8.1016 | −2.4241 | −1.2511 | −0.0700 | −0.0019 | 0.0009 | 0.0037 | 0.0065 |
| 13 | 8.8796 | 7.5879 | 2.4213 | −2.4505 | −1.2379 | −0.0254 | 0.0059 | 0.0044 | 0.0029 | 0.0014 |
| 14 | − 2.2776 | − 2.2746 | − 2.2614 | −2.2738 | −1.1369 | 0 | 0.0072 | 0.0042 | 0.0016 | −0.0010 |
| 15 | −12.1911 | −11.0412 | − 6.4413 | −1.9746 | −0.9322 | 0.0114 | 0.0058 | 0.0032 | 0.0006 | −0.0020 |
| 16 | −21.0445 | −18.9391 | −10.5176 | −1.6105 | −0.8032 | 0.0143 | 0.0036 | 0.0018 | 0.0001 | −0.0016 |
| 17 | −29.3130 | −26.1829 | −13.6624 | −1.2195 | −0.6046 | 0.0126 | 0.0018 | 0.0008 | −0.0001 | −0.0010 |
| 18 | −37.0061 | −32.9694 | −16.8227 | −0.8237 | −0.4073 | 0.0091 | 0.0006 | 0.0002 | −0.0001 | −0.0004 |
| 19 | −44.4785 | −39.4685 | −19.4174 | −0.4325 | −0.2156 | 0.0053 | 0 | 0 | −0.0001 | −0.0002 |
| 20 | −50.3414 | −45.8180 | −22.7243 | −0.0476 | −0.0234 | 0.0017 | −0.0004 | −0.0003 | −0.0001 | 0.0001 |
| 21 | −58.6625 | −52.1155 | −25.9276 | 0.3340 | 0.1666 | −0.0017 | −0.0006 | −0.0003 | 0 | 0.0003 |
| 22 | −62.2043 | −53.2643 | −27.5115 | 0.5248 | 0.2607 | −0.0034 | −0.0007 | −0.0004 | 0 | 0.0004 |
| Area | 55.8133 | 50.1030 | 27.4734 | 5.0166 | 2.7296 | 0.5037 | 0.2521 | 0.1876 | 0.1269 | 0.0382 |

## TABLE 2—INFLUENCE LINES FOR PRESSURE, SECTION 2

| Point | For value of m | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 361.2197 | 213.3197 | 121.5276 | 30.4318 | 17.0256 | 3.6656 | 1.7811 | 1.0905 | 0.8033 | 0.5164 |
| 1 | 224.6273 | 202.6273 | 115.4134 | 28.1995 | 15.5637 | 3.4167 | 1.7082 | 1.2611 | 0.8212 | 0.3824 |
| 2 | 201.6226 | 181.2426 | 102.1214 | 23.7349 | 13.8214 | 2.9188 | 1.5623 | 1.2055 | 0.8563 | 0.5073 |
| 3 | 176.5256 | 159.6555 | 89.0213 | 18.9885 | 10.5722 | 2.0792 | 1.0748 | 0.8182 | 0.5629 | 0.3084 |
| 4 | 153.8833 | 138.4833 | 76.5221 | 14.4406 | 7.8271 | 1.2644 | 0.5915 | 0.4331 | 0.2701 | 0.1109 |
| 5 | 131.6818 | 118.1818 | 64.2110 | 10.3822 | 5.5142 | 0.6330 | 0.2434 | 0.1620 | 0.0816 | 0.0015 |
| 6 | 110.5121 | 99.0701 | 53.0403 | 6.9596 | 3.5141 | 0.2169 | 0.0432 | 0.0182 | −0.0070 | −0.0325 |
| 7 | 91.9127 | 81.3480 | 42.1734 | 4.2153 | 2.1002 | −0.0149 | −0.0452 | −0.0352 | −0.0325 | −0.1009 |
| 8 | 73.2713 | 65.1170 | 33.2001 | 2.1225 | 1.0056 | −0.1153 | −0.0665 | −0.0455 | −0.0290 | −0.1035 |
| 9 | 56.8926 | 50.3967 | 25.1176 | 0.6117 | 0.2379 | −0.1358 | −0.0562 | −0.0362 | −0.0176 | −0.0715 |
| 10 | 41.7214 | 37.1418 | 18.2134 | −0.4072 | −0.2617 | −0.1163 | −0.0368 | −0.0222 | −0.0077 | −0.0376 |
| 11 | 35.2216 | 31.2200 | 15.0317 | −0.7191 | −0.4051 | −0.0996 | −0.0280 | −0.0163 | −0.0049 | −0.0264 |
| 12 | 28.5914 | 25.2983 | 12.0024 | −1.0310 | −0.5564 | −0.0829 | −0.0192 | −0.0105 | −0.0019 | −0.0138 |
| 13 | 16.7735 | 14.6483 | 6.4713 | −1.3520 | −0.7015 | −0.0505 | −0.0071 | −0.0032 | 0.0006 | −0.0020 |
| 14 | 5.9216 | 5.0766 | 2.1133 | −1.4525 | −0.7382 | −0.0253 | −0.0005 | 0.0003 | 0.0012 | 0.0021 |
| 15 | − 4.3323 | − 3.5887 | − 2.5088 | −1.4009 | −0.7047 | −0.0086 | 0.0021 | 0.0015 | 0.0009 | 0.0003 |
| 16 | −12.8220 | −11.5268 | − 6.1344 | −1.2515 | −0.6257 | 0.0006 | 0.0026 | 0.0016 | 0.0005 | −0.0006 |
| 17 | −21.2511 | −18.9100 | − 9.8874 | −1.0444 | −0.5200 | 0.0041 | 0.0020 | 0.0011 | 0.0002 | −0.0007 |
| 18 | −29.6152 | −25.9014 | −12.9386 | −0.8071 | −0.4007 | 0.0056 | 0.0013 | 0.0006 | 0 | −0.0006 |
| 19 | −36.7887 | −32.6444 | −16.5139 | −0.5507 | −0.2729 | 0.0048 | 0.0006 | 0.0003 | 0 | −0.0003 |
| 20 | −44.3112 | −39.2567 | −19.8758 | −0.3024 | −0.1495 | 0.0033 | 0.0001 | 0 | 0 | 0 |
| 21 | −51.2513 | −45.8233 | −22.8137 | −0.0476 | −0.0229 | 0.0017 | −0.0004 | −0.0002 | 0.0001 | 0 |
| 22 | −55.4073 | −49.1071 | −24.1356 | 0.0798 | 0.0400 | 0.0009 | −0.0006 | −0.0001 | 0.0004 | 0.0009 |
| Area | 57.2700 | 50.0635 | 27.4626 | 5.0096 | 2.7774 | 0.5022 | 0.2514 | 0.1862 | 0.1260 | 0.0511 |

## TABLE 3—INFLUENCE LINES FOR PRESSURE, SECTION 3

| Point | For value of m | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 202.8603 | 182.5937 | 101.5273 | 21.7672 | 11.5623 | 1.5847 | 0.5249 | 0.3062 | 0.0878 | −0.1406 |
| 1 | 194.3479 | 174.9479 | 97.4126 | 20.8406 | 11.0451 | 1.7495 | 0.7082 | 0.6355 | 0.2484 | −0.1387 |
| 2 | 177.2653 | 159.6563 | 89.2213 | 18.9884 | 10.5114 | 2.0792 | 1.0748 | 0.9251 | 0.5630 | 0.2014 |
| 3 | 160.1348 | 144.1901 | 80.4113 | 16.9278 | 9.5221 | 2.2339 | 1.2997 | 1.0382 | 0.7782 | 0.5184 |
| 4 | 142.4815 | 128.2155 | 71.1514 | 14.2605 | 8.0446 | 1.8307 | 1.0264 | 0.7972 | 0.5696 | 0.3412 |
| 5 | 124.9362 | 112.2532 | 61.5210 | 11.4189 | 6.3246 | 1.2637 | 0.6383 | 0.4693 | 0.3011 | 0.1334 |
| 6 | 107.7570 | 96.6970 | 52.4662 | 8.6929 | 4.7010 | 0.7430 | 0.3136 | 0.2116 | 0.1093 | 0.0076 |
| 7 | 91.2330 | 81.8287 | 44.2101 | 6.2581 | 3.2522 | 0.3532 | 0.1029 | 0.5192 | 0.0098 | −0.5001 |
| 8 | 75.8302 | 67.8302 | 36.0012 | 4.2033 | 2.1512 | 0.1043 | −0.0060 | −0.0156 | −0.0251 | −0.0350 |
| 9 | 61.1301 | 54.8067 | 28.5114 | 2.5546 | 1.2751 | −0.0289 | −0.0459 | −0.0362 | −0.0272 | −0.0183 |
| 10 | 47.9896 | 42.7919 | 22.0012 | 1.2961 | 0.5075 | −0.0824 | −0.0485 | −0.0331 | −0.0182 | −0.0032 |
| 11 | 41.8436 | 37.2974 | 19.1124 | 0.8412 | 0.3811 | −0.0856 | −0.0423 | −0.0272 | −0.0135 | −0.0001 |
| 12 | 35.7512 | 31.8030 | 16.0101 | 0.3863 | 0.1532 | −0.0889 | −0.0361 | −0.0228 | −0.0089 | 0.0043 |
| 13 | 24.4458 | 21.7011 | 10.7221 | −0.2290 | −0.1501 | −0.0731 | −0.0214 | −0.0124 | −0.0028 | 0.0062 |
| 14 | 14.0391 | 12.4301 | 5.9916 | −0.6083 | −0.3279 | −0.0506 | −0.0098 | −0.0048 | 0.0001 | 0.0050 |
| 15 | 4.4649 | 3.8748 | 1.5143 | −0.8075 | −0.4187 | −0.0299 | −0.0027 | −0.0008 | 0.0010 | 0.0028 |
| 16 | − 4.5192 | − 4.0956 | − 2.4012 | −0.8765 | −0.4454 | −0.0144 | 0.0008 | 0.0009 | 0.0009 | 0.0010 |
| 17 | −12.9577 | −11.6110 | − 6.2242 | −0.8571 | −0.4307 | −0.0044 | 0.0019 | 0.0012 | 0.0006 | 0.0000 |
| 18 | −21.0845 | −18.8005 | − 9.6655 | −0.7823 | −0.3912 | 0.0012 | 0.0018 | 0.0011 | 0.0003 | −0.0005 |
| 19 | −28.8007 | −25.7807 | −13.7176 | −0.6766 | −0.3353 | 0.0038 | 0.0013 | 0.0007 | 0.0001 | −0.0004 |
| 20 | −36.6696 | −32.6496 | −16.5729 | −0.5567 | −0.2762 | 0.0048 | 0.0006 | 0.0003 | −0.0000 | −0.0003 |
| 21 | −44.3791 | −39.4791 | −20.0142 | −0.4324 | −0.2104 | 0.0053 | 0.0000 | 0.0000 | −0.0001 | −0.0002 |
| 22 | −48.2379 | −42.8939 | −21.5176 | −0.3703 | −0.1828 | 0.0055 | −0.0003 | −0.0002 | −0.0002 | −0.0001 |
| Area | 55.6948 | 50.0468 | 27.4289 | 4.9965 | 2.7291 | 0.4977 | 0.2482 | 0.1893 | 0.1233 | 0.0333 |

## TABLE 4—INFLUENCE LINES FOR PRESSURE, SECTION 4

| Point | For value of m | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 171.0648 | 153.6648 | 84.0021 | 14.4936 | 7.3926 | 0.3136 | −0.1000 | −0.1352 | −0.1708 | −0.2064 |
| 1 | 165.4047 | 148.6047 | 81.5013 | 14.4759 | 7.5126 | 0.6346 | 0.1305 | 0.0548 | −0.0209 | −0.0966 |
| 2 | 154.6845 | 138.4845 | 76.1396 | 14.4405 | 7.7993 | 1.2644 | 0.5915 | 0.4308 | 0.2701 | 0.1093 |
| 3 | 142.4858 | 128.2158 | 71.1137 | 14.2605 | 8.0241 | 1.8308 | 1.0264 | 0.7965 | 0.5696 | 0.3427 |
| 4 | 130.5039 | 117.5126 | 65.4033 | 13.6464 | 7.4919 | 2.1438 | 1.2907 | 1.0313 | 0.7776 | 0.5239 |
| 5 | 117.7119 | 105.9588 | 59.0014 | 12.1668 | 7.0510 | 1.8305 | 1.0350 | 0.8014 | 0.5669 | 0.3316 |
| 6 | 104.5525 | 94.0225 | 51.8824 | 10.2532 | 5.7156 | 1.3035 | 0.6512 | 0.4752 | 0.2988 | 0.1222 |
| 7 | 91.2866 | 82.0666 | 45.1736 | 8.2158 | 4.5313 | 0.7924 | 0.3246 | 0.2166 | 0.1078 | −0.0010 |
| 8 | 78.4638 | 70.3538 | 37.8751 | 6.2621 | 3.3144 | 0.3965 | 0.1102 | 0.0556 | 0.0091 | −0.0368 |
| 9 | 65.9745 | 59.0745 | 31.4419 | 4.5177 | 2.3172 | 0.1357 | −0.0022 | −0.0138 | −0.0253 | −0.0368 |
| 10 | 54.1419 | 48.3419 | 25.1033 | 3.0453 | 1.5014 | −0.0096 | −0.0445 | −0.0356 | −0.0272 | −0.0188 |
| 11 | 48.4584 | 43.2984 | 22.6556 | 2.4489 | 1.2717 | −0.0410 | −0.0464 | −0.0342 | −0.0226 | −0.0110 |
| 12 | 42.7850 | 38.2450 | 20.0511 | 1.8525 | 0.8843 | −0.0725 | −0.0484 | −0.0331 | −0.0181 | −0.0031 |

## TABLE 4 (cont.)—INFLUENCE LINES FOR PRESSURE, SECTION 4

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 13 | 32.1841 | 28.7241 | 14.8765 | 0.9565 | 0.4350 | −0.0855 | −0.0365 | −0.0225 | −0.0088 | 0.0079 |
| 14 | 22.2731 | 19.7831 | 10.0010 | 0.2958 | 0.1156 | −0.0733 | −0.0218 | −0.0123 | −0.0027 | 0.0069 |
| 15 | 12.8280 | 11.3641 | 5.5443 | −0.1606 | −0.1055 | −0.0526 | −0.0102 | −0.0051 | 0.0001 | 0.0053 |
| 16 | 3.8659 | 3.3852 | 1.4622 | −0.4567 | −0.2452 | −0.0324 | −0.0029 | −0.0009 | 0.0010 | 0.0029 |
| 17 | − 4.7212 | − 4.2405 | − 2.3176 | −0.6349 | −0.3249 | −0.0165 | 0.0007 | 0.0008 | 0.0009 | 0.0010 |
| 18 | −12.9777 | −11.6077 | − 6.1004 | −0.7329 | −0.3687 | −0.0057 | 0.0019 | 0.0012 | 0.0006 | 0.0000 |
| 19 | −21.1025 | −18.8055 | − 9.6153 | −0.7820 | −0.3901 | 0.0012 | 0.0018 | 0.0010 | 0.0003 | −0.0004 |
| 20 | −29.0817 | −25.9116 | −13.2215 | −0.8070 | −0.4007 | 0.0056 | 0.0013 | 0.0006 | 0.0000 | −0.0006 |
| 21 | −37.1307 | −32.9848 | −16.4013 | −0.8236 | −0.4072 | 0.0092 | 0.0006 | 0.0003 | −0.0001 | −0.0005 |
| 22 | −41.0305 | −36.5216 | −18.5192 | −0.8319 | −0.4104 | 0.0110 | 0.0003 | 0.0001 | −0.0001 | −0.0003 |
| Area | 54.3947 | 50.0365 | 27.4273 | 5.0006 | 2.7138 | 0.4996 | 0.2500 | 0.1873 | 0.1253 | 0.0665 |

## TABLE 5—INFLUENCE LINES FOR PRESSURE, SECTION 5

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 141.6823 | 126.8923 | 67.7172 | 8.6972 | 4.1519 | −0.3133 | −0.3110 | −0.2495 | −0.1887 | −0.1279 |
| 1 | 138.8895 | 123.9895 | 66.4416 | 9.2588 | 4.6322 | 0.0021 | −0.1262 | −0.1124 | −0.0986 | −0.0848 |
| 2 | 131.6439 | 118.1839 | 64.3157 | 10.3821 | 5.4923 | 0.6329 | 0.2434 | 0.1621 | 0.0816 | 0.0011 |
| 3 | 124.8666 | 112.2544 | 61.8055 | 11.4189 | 6.3210 | 1.2636 | 0.6383 | 0.4694 | 0.3011 | 0.1328 |
| 4 | 117.6996 | 105.9596 | 59.0011 | 12.1667 | 6.9826 | 1.8305 | 1.0350 | 0.8012 | 0.5669 | 0.3326 |
| 5 | 109.8046 | 98.9446 | 55.5135 | 12.3152 | 7.2114 | 2.1438 | 1.2824 | 1.0256 | 0.7654 | 0.5052 |
| 6 | 100.6498 | 90.7498 | 51.1516 | 11.4321 | 6.6235 | 1.8306 | 1.0225 | 0.7893 | 0.5559 | 0.3227 |
| 7 | 90.8184 | 81.8178 | 45.8153 | 9.9620 | 5.5718 | 1.3037 | 0.6406 | 0.4655 | 0.2921 | 0.1186 |
| 8 | 80.5377 | 72.4927 | 40.3115 | 8.2352 | 4.4977 | 0.7926 | 0.3176 | 0.2112 | 0.1049 | −0.0014 |
| 9 | 70.1343 | 63.0491 | 34.7084 | 6.4826 | 3.5105 | 0.3966 | 0.1065 | 0.0571 | 0.0084 | −0.0403 |
| 10 | 59.8228 | 53.6808 | 29.1099 | 4.8524 | 2.5219 | 0.1357 | −0.0036 | −0.0141 | −0.0250 | −0.0363 |
| 11 | 54.7816 | 49.1146 | 26.4456 | 4.1402 | 2.1007 | 0.0630 | −0.0241 | −0.0253 | −0.0258 | −0.0361 |
| 12 | 49.6855 | 44.5485 | 24.0016 | 3.4281 | 1.7015 | −0.0097 | −0.0446 | −0.0355 | −0.0267 | −0.0177 |
| 13 | 39.8837 | 35.6737 | 18.8229 | 2.2448 | 1.0095 | −0.0728 | −0.0480 | −0.0325 | −0.0177 | −0.0229 |
| 14 | 30.3676 | 27.1206 | 14.1317 | 1.3032 | 0.6140 | −0.0859 | −0.0360 | −0.0224 | −0.0086 | 0.0052 |
| 15 | 21.1894 | 18.8900 | 9.6925 | 0.5816 | 0.2546 | −0.0739 | −0.0215 | −0.0121 | −0.0026 | 0.0069 |
| 16 | 12.3099 | 10.9499 | 5.5113 | 0.0451 | −0.0494 | −0.0533 | −0.0100 | −0.0049 | 0.0002 | 0.0053 |
| 17 | 3.7084 | 3.2578 | 1.4556 | −0.3466 | −0.1951 | −0.0329 | −0.0029 | −0.0009 | 0.0010 | 0.0029 |
| 18 | − 4.6968 | − 4.2455 | − 2.4401 | −0.6349 | −0.3205 | −0.0165 | 0.0007 | 0.0008 | 0.0009 | 0.0010 |
| 19 | −12.9748 | −11.6208 | − 6.2056 | −0.8570 | −0.4330 | −0.0044 | 0.0019 | 0.0013 | 0.0006 | −0.0001 |
| 20 | −21.1425 | −18.9251 | −10.0553 | −1.0443 | −0.5196 | 0.0047 | 0.0021 | 0.0012 | 0.0002 | −0.0006 |
| 21 | −29.3242 | −26.2032 | −13.7116 | −1.2195 | −0.6010 | 0.0126 | 0.0018 | 0.0009 | 0.0001 | −0.0007 |
| 22 | −33.4219 | −29.8418 | −15.5214 | −1.3071 | −0.6451 | 0.0166 | 0.0016 | 0.0008 | 0.0000 | −0.0008 |
| Area | 55.6773 | 50.0241 | 26.8024 | 5.0072 | 2.7410 | 0.4976 | 0.2481 | 0.1859 | 0.1235 | 0.0612 |

## TABLE 6—INFLUENCE LINES FOR PRESSURE, SECTION 6

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 114.7978 | 102.4818 | 53.2124 | 4.2819 | 1.9496 | −0.3130 | −0.3057 | −0.2111 | −0.0165 | −0.0219 |
| 1 | 113.4055 | 101.3455 | 53.1876 | 5.1744 | 2.4517 | −0.2810 | −0.1894 | −0.1347 | −0.0800 | −0.0253 |
| 2 | 110.5587 | 99.0728 | 53.1289 | 6.9595 | 3.5016 | 0.2169 | 0.0432 | 0.1811 | −0.0070 | −0.1951 |
| 3 | 107.7089 | 96.6989 | 52.6471 | 8.6929 | 4.6504 | 0.7430 | 0.3136 | 0.2114 | 0.1092 | 0.0070 |
| 4 | 104.4980 | 94.0240 | 52.1244 | 10.2531 | 5.7071 | 1.3035 | 0.6512 | 0.4750 | 0.2988 | 0.1226 |
| 5 | 100.3747 | 90.7505 | 51.0536 | 11.4321 | 6.6011 | 1.8307 | 1.0225 | 0.7892 | 0.5559 | 0.3231 |
| 6 | 95.7293 | 86.4878 | 49.5215 | 11.9189 | 7.0042 | 2.1262 | 1.2636 | 1.0092 | 0.7556 | 0.5020 |
| 7 | 89.4129 | 80.7566 | 46.1312 | 11.2884 | 6.5112 | 1.8088 | 1.0065 | 0.7783 | 0.5501 | 0.3221 |
| 8 | 81.9813 | 73.9813 | 41.9876 | 9.9964 | 5.5876 | 1.2846 | 0.6301 | 0.4605 | 0.2895 | 0.1123 |
| 9 | 73.8325 | 66.5205 | 37.2715 | 8.3856 | 4.4906 | 0.7787 | 0.3122 | 0.2083 | 0.1043 | 0.0003 |
| 10 | 65.2027 | 58.6487 | 32.4332 | 6.6989 | 3.5159 | 0.3879 | 0.1044 | 0.0566 | 0.0087 | −0.0392 |
| 11 | 60.7301 | 54.6241 | 30.2011 | 5.8971 | 3.0514 | 0.2595 | 0.0503 | 0.0212 | −0.0079 | −0.0364 |
| 12 | 56.3356 | 50.5996 | 27.6519 | 5.0953 | 2.6098 | 0.1311 | −0.0038 | −0.0142 | −0.0246 | −0.0352 |
| 13 | 47.2771 | 42.4771 | 23.1256 | 3.6668 | 1.8204 | −0.0117 | −0.0441 | −0.0352 | −0.0264 | −0.0176 |
| 14 | 38.4376 | 34.4076 | 18.2906 | 2.4544 | 1.1905 | −0.0734 | −0.0473 | −0.0325 | −0.0176 | −0.0027 |
| 15 | 29.5969 | 26.4519 | 13.8704 | 1.4626 | 0.6972 | −0.0860 | −0.0355 | −0.0220 | −0.0085 | 0.0050 |
| 16 | 20.8770 | 18.6300 | 9.6421 | 0.6711 | 0.3040 | −0.0740 | −0.0213 | −0.0120 | −0.0026 | 0.0068 |
| 17 | 12.3232 | 10.9452 | 5.4419 | 0.0451 | −0.0040 | −0.0533 | −0.0100 | −0.0050 | 0.0002 | 0.0054 |
| 18 | 3.8612 | 3.3755 | 1.4326 | −0.4567 | −0.2445 | −0.0324 | −0.0029 | −0.0009 | 0.0010 | 0.0029 |
| 19 | − 4.5121 | − 4.1101 | − 2.5001 | −0.8764 | −0.4451 | −0.0144 | 0.0008 | 0.0009 | 0.0009 | 0.0009 |
| 20 | −12.8555 | −11.5465 | − 6.3108 | −1.2515 | −0.6261 | 0.0007 | 0.0026 | 0.0016 | 0.0005 | −0.0006 |
| 21 | −21.0495 | −18.9639 | −10.6214 | −1.6104 | −0.7980 | 0.0143 | 0.0036 | 0.0018 | 0.0001 | −0.0016 |
| 22 | −25.3126 | −22.6726 | −12.1216 | −1.7906 | −0.8852 | 0.0211 | 0.0041 | 0.0020 | −0.0001 | −0.0022 |
| Area | 55.6395 | 50.0178 | 27.4832 | 4.8267 | 2.7249 | 0.5073 | 0.2505 | 0.1992 | 0.1264 | 0.0742 |

## TABLE 7—INFLUENCE LINES FOR PRESSURE, SECTION 7

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 90.4317 | 80.5117 | 40.8110 | 1.1190 | 0.3130 | −0.5147 | −0.2192 | −0.1415 | −0.0643 | 0.0129 |
| 1 | 90.5918 | 80.7918 | 41.4868 | 2.1510 | 0.9056 | −0.3481 | −0.1612 | −0.1063 | −0.0534 | −0.0005 |
| 2 | 91.2596 | 81.3516 | 42.7176 | 4.2153 | 2.1007 | −0.0149 | −0.0452 | −0.0386 | −0.0325 | −0.0264 |
| 3 | 91.7857 | 81.8306 | 44.0102 | 6.2182 | 3.2754 | 0.3532 | 0.1029 | 0.0558 | 0.0098 | −0.0356 |
| 4 | 92.5503 | 82.0673 | 45.1315 | 8.2157 | 4.4916 | 0.7924 | 0.3247 | 0.2162 | 0.1078 | −0.0006 |
| 5 | 90.8173 | 81.8173 | 45.8056 | 9.9620 | 5.6055 | 1.3037 | 0.6406 | 0.4662 | 0.2921 | 0.1180 |
| 6 | 89.4423 | 80.7550 | 46.0056 | 11.2884 | 6.5406 | 1.8088 | 1.0065 | 0.7782 | 0.5500 | 0.3221 |
| 7 | 86.8007 | 78.4752 | 45.1730 | 11.8870 | 7.4176 | 2.0991 | 1.2500 | 1.0009 | 0.7519 | 0.5029 |
| 8 | 82.3717 | 74.4817 | 42.9090 | 11.3368 | 6.5499 | 1.7851 | 0.9976 | 0.7726 | 0.5484 | 0.3242 |
| 9 | 76.6072 | 69.2154 | 39.6480 | 10.0982 | 5.6626 | 1.2674 | 0.6254 | 0.4571 | 0.2891 | 0.1232 |
| 10 | 72.4322 | 63.0322 | 35.5012 | 8.5178 | 4.6339 | 0.7680 | 0.3104 | 0.2075 | 0.1044 | 0.0016 |
| 11 | 66.2722 | 59.6360 | 33.1011 | 7.6797 | 4.1274 | 0.5751 | 0.2073 | 0.1320 | 0.0567 | −0.0186 |
| 12 | 62.4246 | 56.2398 | 31.5006 | 6.8417 | 3.6120 | 0.3822 | 0.1043 | 0.0566 | 0.0089 | −0.0388 |
| 13 | 55.7775 | 49.0263 | 27.1215 | 5.2306 | 2.7712 | 0.1281 | −0.0033 | −0.0138 | −0.0244 | −0.0350 |
| 14 | 46.2942 | 41.5802 | 22.7243 | 3.7767 | 1.8809 | −0.0124 | −0.0436 | −0.0351 | −0.0263 | −0.0175 |
| 15 | 37.9801 | 34.0304 | 18.2513 | 2.5199 | 1.2212 | −0.0735 | −0.0470 | −0.0324 | −0.0175 | −0.0026 |
| 16 | 29.5823 | 26.4457 | 13.8991 | 1.4626 | 0.6883 | −0.0860 | −0.0355 | −0.0220 | −0.0085 | 0.0050 |
| 17 | 21.1739 | 18.8793 | 9.7008 | 0.5816 | 0.2538 | −0.0739 | −0.0215 | −0.0120 | −0.0026 | 0.0068 |
| 18 | 12.8351 | 11.3489 | 5.5941 | −0.1606 | −0.1066 | −0.0526 | −0.0102 | −0.0051 | 0.0001 | 0.0053 |
| 19 | 4.4406 | 3.8550 | 1.5126 | −0.8075 | −0.4187 | −0.0299 | 0.0027 | 0.0018 | 0.0010 | 0.0002 |
| 20 | − 3.8897 | − 3.6132 | − 2.5070 | −1.4008 | −0.7048 | −0.0087 | 0.0021 | 0.0015 | 0.0009 | 0.0003 |
| 21 | −12.2099 | −11.0704 | − 6.5121 | −1.9744 | −0.9815 | 0.0114 | 0.0058 | 0.0032 | 0.0006 | −0.0020 |
| 22 | −16.6725 | −14.7985 | − 8.3030 | −2.1735 | −1.0760 | 0.0214 | 0.0076 | 0.0038 | 0.0001 | −0.0036 |
| Area | 55.9443 | 50.0145 | 27.4682 | 4.9834 | 2.7700 | 0.4858 | 0.2490 | 0.1864 | 0.1237 | 0.0614 |

## TABLE 8—INFLUENCE LINES FOR PRESSURE, SECTION 8

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 68.6951 | 60.9592 | 30.0155 | −0.9994 | −0.6995 | −0.3991 | −0.1256 | −0.0731 | −0.0206 | 0.0319 |
| 1 | 70.1346 | 62.3466 | 31.1937 | 0.0412 | −0.1316 | −0.3045 | −0.1059 | −0.0646 | −0.0234 | 0.0178 |
| 2 | 72.9964 | 65.1214 | 33.6212 | 2.1224 | 1.0057 | −0.1153 | −0.0665 | −0.0478 | −0.0290 | −0.0102 |
| 3 | 75.7859 | 67.8339 | 36.0156 | 4.2034 | 2.1514 | 0.1043 | −0.0000 | −0.0156 | −0.0251 | −0.0346 |
| 4 | 78.4171 | 70.3571 | 38.1212 | 6.2621 | 3.3293 | 0.3965 | 0.1102 | 0.0596 | 0.0091 | −0.0414 |
| 5 | 80.5655 | 72.4953 | 40.2143 | 8.2353 | 4.4997 | 0.7926 | 0.3177 | 0.2113 | 0.1049 | −0.0015 |
| 6 | 82.0108 | 73.9832 | 41.8726 | 9.9964 | 5.5172 | 1.2846 | 0.6301 | 0.4598 | 0.2895 | 0.1192 |
| 7 | 83.6286 | 74.4854 | 42.9124 | 11.3369 | 6.5002 | 1.7851 | 0.9976 | 0.7730 | 0.5484 | 0.3238 |
| 8 | 81.3119 | 73.5818 | 42.6612 | 11.9480 | 7.0105 | 2.0784 | 1.2442 | 0.9977 | 0.7512 | 0.5047 |
| 9 | 78.1292 | 70.7942 | 41.0916 | 11.4080 | 6.5223 | 1.7701 | 0.9945 | 0.7713 | 0.5482 | 0.3251 |
| 10 | 73.6234 | 66.5594 | 38.3041 | 10.1753 | 5.6878 | 1.2580 | 0.6243 | 0.4568 | 0.2892 | 0.1216 |
| 11 | 70.7593 | 63.9097 | 36.5113 | 9.3898 | 5.2022 | 1.0109 | 0.4673 | 0.3321 | 0.1968 | 0.0615 |
| 12 | 67.8573 | 61.2600 | 34.8707 | 8.5944 | 4.6423 | 0.7630 | 0.3103 | 0.2088 | 0.1045 | 0.0002 |
| 13 | 61.1991 | 55.1706 | 31.0566 | 6.9080 | 3.6392 | 0.3801 | 0.1045 | 0.0568 | 0.0090 | −0.0388 |
| 14 | 53.9537 | 48.5475 | 26.9214 | 5.2729 | 2.7105 | 0.1281 | −0.0031 | −0.0138 | −0.0244 | −0.0350 |
| 15 | 46.3100 | 41.5796 | 22.6577 | 3.7767 | 1.8501 | −0.0723 | −0.0436 | −0.0349 | −0.0263 | −0.0177 |
| 16 | 38.3981 | 34.3985 | 18.3992 | 2.4544 | 1.1956 | −0.0734 | −0.0473 | −0.0324 | −0.0176 | −0.0028 |
| 17 | 30.3362 | 27.1069 | 14.1896 | 1.3032 | 0.6071 | −0.0859 | −0.0360 | −0.0223 | −0.0086 | 0.0051 |
| 18 | 22.1679 | 19.7647 | 10.0516 | 0.2958 | 0.1112 | −0.0733 | −0.0218 | −0.0123 | −0.0027 | 0.0069 |
| 19 | 14.0269 | 12.4069 | 5.9058 | −0.6082 | −0.3290 | −0.0506 | −0.0098 | −0.0048 | 0.0001 | 0.0050 |
| 20 | 5.8621 | 5.0487 | 1.7951 | −1.4523 | −0.7388 | −0.0253 | −0.0005 | 0.0004 | 0.0012 | 0.0020 |
| 21 | − 2.5614 | − 2.3072 | − 1.2905 | −2.2736 | −1.1368 | 0.0000 | 0.0072 | 0.0044 | 0.0016 | −0.0012 |
| 22 | − 6.3637 | − 5.9551 | − 4.3204 | −2.6842 | −1.3358 | 0.0126 | 0.0105 | 0.0062 | 0.0018 | −0.0026 |
| Area | 54.9323 | 50.0070 | 27.4993 | 4.9950 | 2.7296 | 0.4967 | 0.2509 | 0.1886 | 0.1245 | 0.0635 |

## TABLE 9—INFLUENCE LINES FOR PRESSURE, SECTION 9

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 49.4722 | 43.7192 | 20.7070 | −2.2827 | −1.2300 | −0.1773 | −0.0545 | −0.0274 | 0.0002 | 0.0278 |
| 1 | 51.8558 | 45.9468 | 22.3113 | −1.3179 | −0.7612 | −0.2207 | −0.0554 | −0.0306 | −0.0057 | 0.0192 |
| 2 | 56.7782 | 50.4020 | 24.8950 | 0.6117 | +0.2379 | −0.1358 | −0.0562 | −0.0369 | −0.0176 | 0.0017 |
| 3 | 61.3438 | 54.8114 | 28.6818 | 2.5546 | 1.2644 | −0.0289 | −0.0459 | −0.0366 | −0.0272 | −0.0178 |
| 4 | 65.8989 | 59.0788 | 31.7982 | 4.5177 | 2.3267 | 0.1357 | −0.0022 | −0.0138 | −0.0253 | −0.0368 |
| 5 | 70.1428 | 63.0528 | 34.7001 | 6.4826 | 3.4391 | 0.3966 | 0.1065 | 0.0574 | 0.0084 | −0.0406 |
| 6 | 73.7292 | 66.5234 | 37.5014 | 8.3856 | 4.5821 | 0.7787 | 0.3122 | 0.2082 | 0.1043 | 0.0004 |
| 7 | 76.6125 | 69.2203 | 39.6513 | 10.0982 | 5.6822 | 1.2674 | 0.6254 | 0.4572 | 0.2891 | 0.1210 |
| 8 | 78.2187 | 70.7954 | 41.1021 | 11.4080 | 6.5899 | 1.7401 | 0.9945 | 0.7714 | 0.5483 | 0.3252 |
| 9 | 78.2090 | 70.8479 | 41.4034 | 12.0018 | 7.0101 | 2.0675 | 1.2426 | 0.9969 | 0.7512 | 0.5055 |
| 10 | 76.0761 | 68.8955 | 40.1732 | 11.4521 | 6.2437 | 1.7633 | 0.9939 | 0.7761 | 0.5483 | 0.3205 |
| 11 | 74.1852 | 67.1452 | 38.9880 | 10.8318 | 6.1514 | 1.5068 | 0.8090 | 0.6138 | 0.4187 | 0.2236 |
| 12 | 72.2950 | 65.3950 | 37.8030 | 10.2116 | 5.7231 | 1.2546 | 0.6242 | 0.4567 | 0.2892 | 0.1217 |

## TABLE 9 (cont.)—INFLUENCE LINES FOR PRESSURE, SECTION 9

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 13 | 67.2220 | 60.7104 | 34.6641 | 8.6183 | 4.6818 | 0.7618 | 0.3104 | 0.2074 | 0.1045 | 0.0016 |
| 14 | 61.2046 | 55.1662 | 31.0304 | 6.9080 | 3.6422 | 0.3802 | 0.1045 | 0.0568 | 0.0090 | −0.0388 |
| 15 | 54.4959 | 49.0198 | 27.1252 | 5.2306 | 2.6796 | 0.1287 | −0.0033 | −0.0138 | −0.0244 | −0.0350 |
| 16 | 47.3134 | 42.4638 | 23.0653 | 3.6668 | 1.8209 | −0.0116 | −0.0441 | −0.0353 | −0.0264 | −0.0175 |
| 17 | 39.8323 | 35.6559 | 18.9501 | 2.2449 | 1.0864 | −0.0728 | −0.0480 | −0.0328 | −0.0177 | −0.0026 |
| 18 | 32.1717 | 28.7018 | 14.8221 | 0.9565 | 0.4355 | −0.0855 | −0.0365 | −0.0226 | −0.0088 | 0.0050 |
| 19 | 24.4148 | 21.6742 | 10.7116 | −0.2289 | −0.1501 | −0.0731 | −0.0214 | −0.0121 | −0.0028 | 0.0065 |
| 20 | 16.6169 | 14.6169 | 6.6303 | −1.3519 | −0.7007 | −0.0505 | −0.0071 | −0.0033 | 0.0006 | 0.0045 |
| 21 | 8.8025 | 7.5521 | 2.5502 | −2.4503 | −1.2351 | −0.0254 | 0.0059 | 0.0044 | 0.0029 | 0.0014 |
| 22 | 4.9198 | 4.0197 | 0.5098 | −3.0010 | −1.5194 | −0.0379 | 0.0124 | 0.0082 | 0.0041 | 0.0000 |
| Area | 55.6356 | 50.0014 | 27.4544 | 4.9926 | 2.7203 | 0.4976 | 0.2489 | 0.1868 | 0.1240 | 0.0611 |

## TABLE 10—INFLUENCE LINES FOR PRESSURE, SECTION 10

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 32.5730 | 28.6266 | 12.8410 | −2.9309 | −1.5306 | −0.1466 | −0.0130 | −0.0032 | 0.0067 | 0.0166 |
| 1 | 35.6611 | 31.4671 | 14.6919 | −2.0897 | −1.1131 | −0.1365 | −0.0209 | −0.0095 | 0.0019 | 0.0133 |
| 2 | 41.8482 | 37.1482 | 18.3711 | −0.4072 | −0.2606 | −0.1163 | −0.0368 | −0.0223 | −0.0077 | 0.0069 |
| 3 | 47.9878 | 42.7978 | 22.0403 | 1.2961 | 0.6440 | −0.0824 | −0.0485 | −0.0334 | −0.0182 | −0.0030 |
| 4 | 54.0117 | 48.3475 | 25.6908 | 3.0453 | 1.5178 | −0.0096 | −0.0445 | −0.0358 | −0.0272 | −0.0186 |
| 5 | 59.7662 | 53.6858 | 29.3641 | 4.8524 | 2.5002 | 0.1357 | −0.0036 | −0.0143 | −0.0250 | −0.0357 |
| 6 | 65.0487 | 58.6531 | 32.6708 | 6.6989 | 3.5014 | 0.3879 | 0.1044 | 0.0566 | 0.0087 | −0.0392 |
| 7 | 69.9185 | 63.0385 | 35.7669 | 8.5179 | 4.6113 | 0.7680 | 0.3104 | 0.2074 | 0.1044 | 0.0014 |
| 8 | 73.5622 | 66.5622 | 38.2918 | 10.1753 | 5.7134 | 1.2580 | 0.6243 | 0.4556 | 0.2892 | 0.1216 |
| 9 | 76.0970 | 68.8970 | 40.1816 | 11.4522 | 6.6077 | 1.7633 | 0.9939 | 0.7693 | 0.5483 | 0.3273 |
| 10 | 76.8386 | 69.6386 | 40.8303 | 12.0275 | 7.0450 | 2.0635 | 1.2424 | 0.9968 | 0.7512 | 0.5056 |
| 11 | 76.1285 | 68.9785 | 40.3611 | 11.7467 | 6.8291 | 1.9126 | 1.1181 | 0.8840 | 0.6498 | 0.4156 |
| 12 | 75.4185 | 68.3185 | 39.8898 | 11.4659 | 6.6123 | 1.7616 | 0.9939 | 0.7711 | 0.5483 | 0.3255 |
| 13 | 72.3185 | 65.3909 | 37.8001 | 10.2116 | 5.7331 | 1.2546 | 0.6242 | 0.4567 | 0.2892 | 0.1217 |
| 14 | 67.8516 | 61.2516 | 34.9212 | 8.5944 | 4.6707 | 0.7630 | 0.3104 | 0.2074 | 0.1045 | 0.0016 |
| 15 | 62.3995 | 26.2295 | 31.5303 | 6.8418 | 3.6156 | 0.3822 | 0.1043 | 0.0566 | 0.0089 | −0.0398 |
| 16 | 56.2725 | 50.5825 | 27.8314 | 5.0953 | 2.6055 | 0.1311 | −0.0038 | −0.0142 | −0.0246 | −0.0350 |
| 17 | 49.6671 | 44.5271 | 23.9714 | 3.4282 | 1.7092 | −0.0097 | −0.0446 | −0.0357 | −0.0267 | −0.0177 |
| 18 | 42.7193 | 38.2193 | 20.0414 | 1.8626 | 0.9001 | −0.0725 | −0.0484 | −0.0333 | −0.0181 | −0.0029 |
| 19 | 35.6740 | 31.7730 | 16.1112 | 0.3865 | 0.1505 | −0.0889 | −0.0361 | −0.0225 | −0.0089 | 0.0047 |
| 20 | 28.5653 | 25.2641 | 12.0192 | −1.0308 | −0.5510 | −0.0830 | −0.0192 | −0.0105 | −0.0018 | 0.0069 |
| 21 | 21.2977 | 18.7364 | 8.4912 | −2.4239 | −1.2411 | −0.0700 | −0.0019 | 0.0009 | 0.0037 | 0.0065 |
| 22 | 17.7966 | 15.4726 | 6.1766 | −3.1194 | −1.5913 | −0.0635 | 0.0067 | 0.0066 | 0.0065 | 0.0064 |
| Area | 55.6104 | 48.0020 | 27.5045 | 4.9976 | 2.8583 | 0.5010 | 0.2512 | 0.1885 | 0.1260 | 0.0634 |

## TABLE 11—INFLUENCE LINES FOR PRESSURE, SECTION 11

| Point | For value of $m$ | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 0.0001 | 0.001 | 0.005 | 0.01 | 0.05 | 0.10 | 0.20 | 0.30 | 0.40 | 0.50 |
| 0 | 25.2817 | 22.0497 | 9.1215 | −3.7936 | −1.9423 | −0.1050 | −0.0003 | −0.0011 | −0.0019 | −0.0027 |
| 1 | 28.5128 | 25.1018 | 11.4173 | −2.2568 | −1.1816 | −0.1032 | −0.0113 | −0.0066 | −0.0028 | 0.0010 |
| 2 | 35.1972 | 31.2060 | 15.2410 | −0.7190 | −0.4090 | −0.0995 | −0.0279 | −0.0162 | −0.0047 | 0.0068 |
| 3 | 42.0512 | 37.2853 | 18.2214 | 0.8412 | 0.3816 | −0.0856 | −0.0373 | −0.0300 | −0.0135 | 0.0030 |
| 4 | 49.0003 | 43.2834 | 20.4156 | 2.4538 | 1.2064 | −0.0410 | −0.0464 | −0.0343 | −0.0226 | −0.0109 |
| 5 | 54.7341 | 49.1064 | 26.5916 | 4.1402 | 2.1050 | 0.0630 | −0.0240 | −0.0252 | −0.0258 | −0.0264 |
| 6 | 60.7093 | 54.6117 | 30.2211 | 5.8971 | 3.0711 | 0.2595 | 0.0503 | 0.0214 | −0.0080 | −0.0374 |
| 7 | 66.1297 | 59.6340 | 33.6513 | 7.6798 | 4.1273 | 0.5750 | 0.2073 | 0.1320 | 0.0567 | −0.0186 |
| 8 | 70.4831 | 63.9069 | 36.5998 | 9.3898 | 5.1822 | 1.0105 | 0.4673 | 0.3320 | 0.1968 | 0.0616 |
| 9 | 74.1853 | 67.1440 | 38.9787 | 10.8318 | 6.1703 | 1.5089 | 0.8090 | 0.6138 | 0.4187 | 0.2632 |
| 10 | 76.1427 | 68.9785 | 40.3218 | 11.7467 | 6.8202 | 1.9124 | 1.1181 | 0.8839 | 0.6497 | 0.4155 |
| 11 | 77.5517 | 69.8957 | 41.2726 | 12.6616 | 7.4655 | 2.3159 | 1.4272 | 1.0892 | 0.8712 | 0.6532 |
| 12 | | | | | | | | | | |
| 13 | | | | | | | | | | |
| 14 | | | | | | | | | | |
| 15 | | | | | | | | | | |
| 16 | | | | | | | | | | |
| 17 | | | R | E | P | E | A | T | | |
| 18 | | | | | | | | | | |
| 19 | | | | | | | | | | |
| 20 | | | | | | | | | | |
| 21 | | | | | | | | | | |
| 22 | | | | | | | | | | |
| Area | 55.6882 | 50.1701 | 27.1403 | 4.9749 | 2.8473 | 0.5000 | 0.2507 | 0.1872 | 0.1244 | 0.0657 |

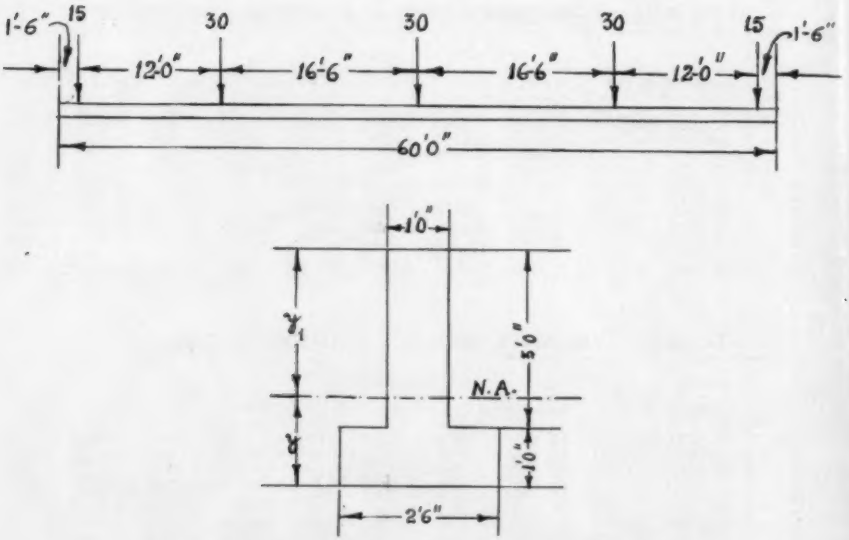



Fig. 3—Dimensions and loading of beam used in example

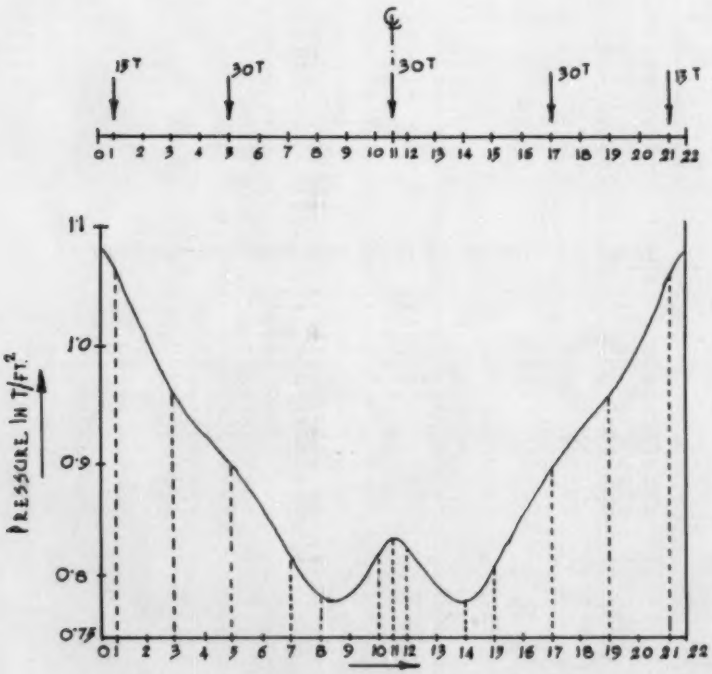



Fig. 4—Pressure diagram worked out in the example

*Continued from p. 732*

$$I = \frac{2.4^3 + 2.5 \times 1.6^3 - 1.5 \times 0.6^3}{3} = 7.82 \text{ ft}^4$$

$$m = \frac{Kb}{EI}\left(\frac{L}{20}\right)^4 = \frac{100 \times 30 \times 36^4}{3 \times 10^6 \times 7.82 \times 12^4} = 0.01$$

Points 1 and 21, 3 and 19, 5 and 17, 7 and 15, 8 and 14 have identical influence lines; point 11 is the center line of the beam. Pressures at all these points are calculated and plotted (Fig. 4). Points 2, 4, etc. have been omitted from investigation to avoid bulk in calculations.

Load points and standard sections are shown in Fig. 3 and 4. It may be seen that load points have been so chosen so as to coincide with any of the standard sections. This has been done for the sake of convenience. For other load positions the values of ordinates under load may easily be interpolated from the tables. But it is desirable to arrive at the former situation by selecting the length of the beam. The sections are chosen as 1/20th of the length of the beam so that this can be very easily accomplished. Results for shear and moment are plotted in Fig. 5 and 6.

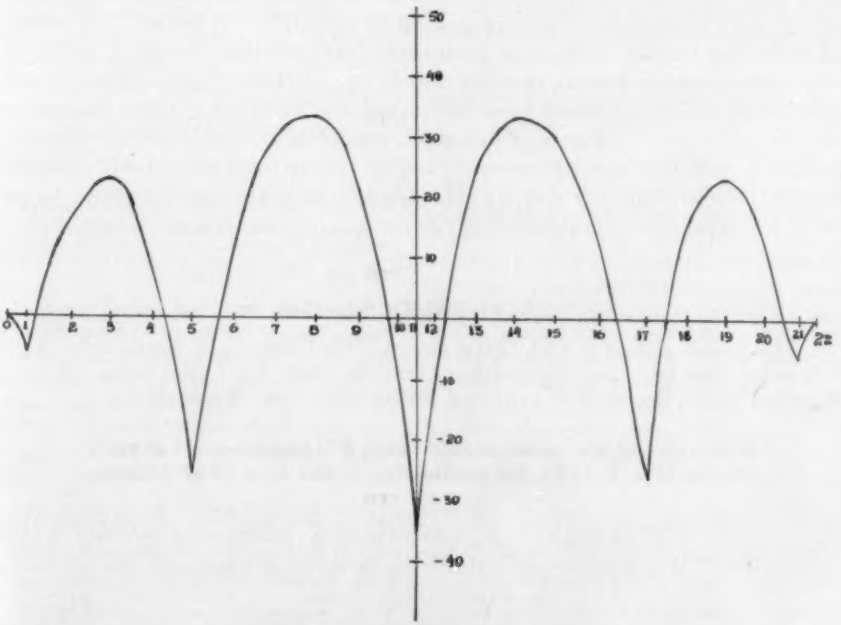



**Fig. 5—Bending moment diagram**

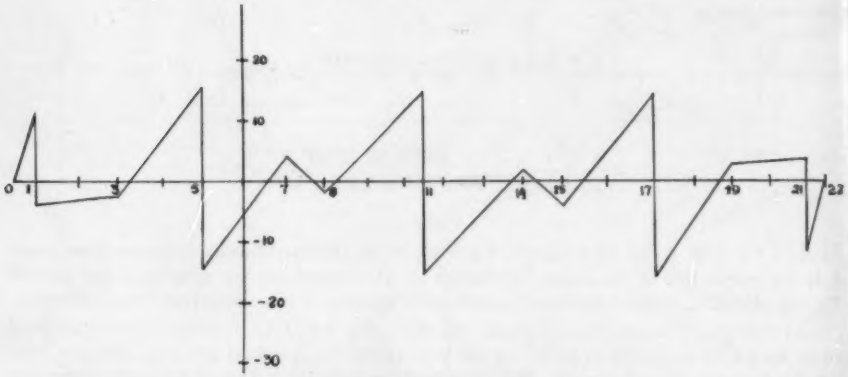



Fig. 6—Shear diagram

## CONCLUSION

The influence line method is very handy and is specially useful in working out dynamic load effects. An experimental analysis was done by photoelastic method with a single load on a prepared subgrade. Values of deflection checked admirably with those provided in the paper. The experimental beam and subgrade were of such dimensions as to give a value of $m = 0.00005$, which may be considered as approaching an infinitely stiff beam. The values of deflection for this value of $m$ were compared with those for $m = 0.0001$ in the tables and the results checked within two decimal places. This proved that below a certain critical value of $m$ which can be taken as 0.001 the results become asymptotic. Values of influence lines for $m = 0.0001$ may be used for infinitely stiff footings. Some very heavy foundations have been designed on this principle recently and are considered to be behaving admirably. Great economy in sections was obtained in this method over the conventional methods of design.

## ACKNOWLEDGMENTS

I am extremely grateful to Prof. Gerald Pickett of the University of Wisconsin for all the help he has given me in working out the problem. My students Sarvashri Sudhin Gupta, S. Badaruddin, B. P. Kapoor, S. Sarkar, and S. Tsering rendered me all possible help.

**Discussion of this paper should reach ACI headquarters in triplicate by Mar. 1, 1959, for publication in the June 1959 JOURNAL.**

# CURRENT REVIEWS

## of Significant Contributions in Foreign and Domestic Publications

## Bridges

### I-shaped beams of reinforced concrete in bridges (Les poutres profilées en béton armé dans les ponts)

EDGAR GILBERT, *L'Ingénieur* (Montreal), V. 43, No. 171, Autumn 1957, pp. 13-19.
Reviewed by ARON L. MIRSKY

A broad study of the design of concrete highway bridges with I-shaped girders, covering such practical elements as creep, shear and bond, placing sequences, etc.

### Widening of the Montagnac Bridge over the Herault (Élargissement du Pont de Montagnac sur l'Hérault)

A. BESOMBES-VAILHE, *Annales des Ponts et Chaussées* (Paris), V. 127, No. 5, Sept.-Oct. 1957, pp. 617-628
Reviewed by ARON L. MIRSKY

Seventeenth century stone arch structure on French National Highway 113 was widened by removing old superstructure and constructing reinforced concrete deck, cantilevering out on either side of the old arches. Architectural as well as engineering problems were surmounted by this solution, which with necessary variations may be used for other projects of this type.

### Langstone Bridge, Hampshire

JOHN W. MELROSE and WALTER A. EYRE, *Proceedings*, Institution of Civil Engineers (London), V. 8 (Session 1956-57), Oct. 1957, pp. 141-160
Reviewed by ARON L. MIRSKY

Design of bridge, one of few toll bridges of any importance remaining in the United Kingdom, was governed by statutory requirements and need for rapid construction. Aesthetics and economy were also factors in selection of simple 29-span structure of reinforced-concrete pile bents supporting a prestressed concrete deck formed of precast beams transversely stressed together after placing. Casting yard was located at the site.

Discussion is printed in issue of April 1958, pp. 514-540.

## Construction

### Design and construction of a dry dock at South Shields for Messrs. Brigham and Cowan, Ltd.

PETER F. STOTT and LANES M. RAMAGE, *Proceedings*, Institution of Civil Engineers (London), V. 8 (Session 1956-57), Oct. 1957, pp. 161-188; discussion, Apr. 1958 issue, pp. 541-555
Reviewed by ARON L. MIRSKY

Interesting description of layout, design, and construction of drydock capable of accommodating a 38,000-ton tanker. Full ancillary works (riverside jetties, sludge-disposal tanks, etc.) were included in the project.

### Bron-Parilly housing development for 2600 families, industrialized construction (in French)

P. BOURDEIX, R. GAGES, F. GRIMAL, and H. MOISE, Supplement to *Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), No. 127-128, July-Aug. 1958, Series Technique Gènèrale de la Construction (23), pp. 834-860
AUTHORS' SUMMARY

The Bron-Parilly housing development providing lodging for 2600 families includes a 15-story building, four towers, seven 6-story buildings. In addition there are four school buildings, an administrative center, a business center, a market, a parish center, and a community garage. The development was designed in terms of functional units adapted for construction on an industrial basis.

The structural system of the development is based on concrete bearing walls supporting reinforced concrete floor slabs 6 in. thick. Metal forming was used throughout with

A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 30, No. 6, Dec. 1958 (*Proceedings* V. 54). Address P.O. Box 4754, Redford Station, Detroit 19, Mich. Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are **not** available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

welded steel reinforcing mats and truck-mounted concrete mixers. The front façade is a prefabricated unit including glazed sections and flush metal panels enclosing a hollow space filled with "bakelised" mineral wool insulation.

### No. 1 Berth, Tilbury Dock

CECIL PEEL, ARTHUR J. CARMICHAEL, and RONALD F. J. SMEARDON, *Proceedings*, Institution of Civil Engineers (London), V. 8 (session 1957-58), Dec. 1957, pp. 331-362; discussion, June 1958 issue, pp. 200-211

Reviewed by ARON L. MIRSKY

New berths added to Tilbury Dock, constructed in 1886 and enlarged several times since, feature a new quay 842 ft long and a T-shaped transit shed. Quay wall was constructed of concrete monoliths 30 ft square, sunk to depths between 60 and 81 ft below coping level and surmounted by a cantilevered quay 50 ft wide. Shed roof is entirely of barrel vaults to obviate interior columns. Appendices outline analysis of monolith stability and design of barrel-vault roofs.

## Construction Techniques

### Guiding principles for the field control of concrete and reinforced concrete (Leitsätze für die bauüberwachung im beton und stahlbetonbau)

WALTER NAKONZ, Editor, Deutscher Beton-Verein E. V., Wiesbaden, 10th Edition, 1957, 156 pp.

Reviewed by J. F. LEPPMANN

Rules, standards, and recommended practice for field personnel, covering the selection of materials, the supervision of operations, and the preparation of tests, brought up-to-date as of July 1957. The sections on bending and welding of steel, on open grade mixes, and on consistency tests have been enlarged. Various charts and pictures and a section on hammer blow tests have been added. Steam curing and prestressing practices are not included.

## Design

### Shear rupture in reinforced concrete beams (in Danish)

MOGENS FREDERIKSEN, *Ingeniøren*, B Edition (Copenhagen), V. 67, No. 12, June 1958, pp. 388-393

Reviewed by JESPER STRANDGAARD

The author attempts to establish a theoretical analysis for the shear design of reinforced concrete beams based on the stress-strain curves and Mohr's rupture envelope for concrete. A simply supported beam with no compression reinforcement was analyzed, and the form and location of the principal fissure due to diagonal tension was derived mathematically. The developed formulas were applied to four of the tests described by Moody, Viest, Elstner, and Hognestad in "Shear Strength of Reinforced Concrete Beams," ACI JOURNAL, V. 26, No. 4, Dec. 1954, and the theoretical shear failure was found to agree well with the test results. Author stresses the need for further research.

### Stresses in tunnel cuts with rectangular pay-section and arch-shaped roof (Spannungen an Tunnelöffnungen mit rechteckigem Nutzquerschnitt und Kreisbogenförmiger Überwölbung)

R. HILTSCHER, *Der Bauingenieur* (Berlin), V. 32, No. 8, Aug. 1957, pp. 288-291

Reviewed by ARON L. MIRSKY

Results of photoelastic investigations, presented in form of charts using dimensionless parameters (e.g. $\sigma/p$, $h/b$). Author notes that for design of reinforcement, additional data are required (see following review).

### Total tensile force on openings in a uniaxial compressive stress field (Die totale Zugkraft an Öffnungen in einem einachsigen Druckspannungsfeld)

R. HILTSCHER and B. PANT, *Der Bauingenieur* (Berlin), V. 32, No. 12, Dec. 1957, pp. 470-474

Reviewed by ARON L. MIRSKY

Investigation, photoelastic and analytic, of variously-shaped holes, with application to design of reinforcement around tunnels in rock. (See preceding review.)

### Analysis of grids of crossed beams (Calcul des réseaux de poutres croisées)

JEAN COURBON and ROGER LACROIX, *Annales des Ponts et Chaussées* (Paris), V. 127, 1957: No. 3, May-June, pp. 271-318; No. 4, July-Aug., pp. 391-444

Reviewed by ARON L. MIRSKY

Method, claimed to be exact, reduces analysis of grid to analysis of a continuous beam on elastic supports. Loads applied to the grid are resolved into a linear combination of a finite number of "proper loads" such that under their action the beams of one of the systems comprising the grid are distorted proportionally to the effect of the actual loads. If the beams composing the grid are

equally spaced and of constant moment of inertia, these "proper loads" are known, their resolution is immediately effected, and the solution of a system of equations is not necessary. The method can be generalized to include the case wherein the beams of one system are infinitesimal distances apart, i.e., slabs. Matrix analysis is used extensively. Tables to simplify calculations, and worked examples are included.

## Materials

### Influence of free lime in portland cement on the resistance of concrete against aggressive actions (Influencia de la cal libre en el cemento portland sobre la resistencia del hormigón frente a las acciones agresivas)

F. W. Meier-Grolman, *Bulletin* No. 182, Instituto Técnico de la Construcción y del Cemento, Madrid, 1957, 72 pp.

Reviewed by Roger Diaz de Cossio

The results of long term research on American portland cement when subjected to the action of sulfate are analyzed and interpreted. A new definition is given of "free hydrate of lime." It is found that the free lime content must not exceed 0.5 percent for lean mixes, and that the limiting value for tricalcium aluminate must be 6 percent. In fact, all essential ASTM specifications on Type V cements are confirmed. A modulus of aggressiveness is offered as the ratio between the chemically reacting elements and those reacting in the liquid phase. It varies inversely with the free lime content of cement.

### Influence of fineness on the properties of cement (Influencia de la finura de los cementos en sus propiedades)

Adrian Margarit Duran, *Bulletin* No. 167, Instituto Técnico de la Construcción y del Cemento, Madrid, 57 pp.

Reviewed by Roger Diaz de Cossio

Reports results of an investigation conducted during the past few years at the Barcelona Laboratories of the Institute. Using ten commercial Spanish portland cements and their respective clinkers, the influence of the fineness of cement was considered on the following factors: real and apparent specific weight, mix water percentage, setting time, tensile and compressive strengths at 7 and 28 days, chemical composition, and age. The fineness was measured by several different methods and general fineness coefficients were then adopted. In general, it was found that the fineness had a decisive effect on the over-all quality of cement; also, none of the usual methods to determine cement fineness was found completely satisfactory. Consequently a new method to determine the specific surface of cement was developed. The method is based on the capacity of cement to absorb coloring substances while in the liquid state.

### Effects of the addition of salts on the false set of cements

Hiroshi Narahashi, Masashi Amemiya, and Kokichi Nakashima, *Semento Gijutsu Nenpo*, V. 11, 1958, pp. 66-75

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

The effects of the addition of 0.5 percent by weight of various chlorides, sulfates, or nitrates on the relaxation of false set were studied by the penetration method and the flow test of mortar. In the scope of the experiments, $BaCl_2$ and $AlCl_3$ seem to be effective.

### Effect of the temperature of materials on the properties of cement paste and mortar

Kiyoshi Miyazawa, *Semento Gijutsu Nenpo*, V. 11, 1957, pp. 85-89

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

Tests were made in a constant temperature room at 20 ± 1 C on materials of various temperatures. The effect of temperature of the materials on the properties of cement paste was small. The flow of mortar and compressive strength at 3 days increased with rising temperature of materials.

### Effect of calcium chloride on sulfate resistance of mortar

Nobue Fukuda, Nobuo Adachi, and Iwao Yamada, *Semento Gijutsu Nenpo*, V. 11, 1957, pp. 79-85

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

After removal from the mold at 1 day, mortar specimens (water:cement:sand ratio 0.65:1:2) were cured at 20 ± 1 C for 7, 15, and 30 days in water or in a moist closet of 94 to 98 percent relative humidity and then immersed in 10 percent solution of

$Na_2SO_4$ or $MgSO_4$. The effect of the initial curing conditions of mortar containing 0 to 3 percent anhydrous $CaCl_2$ by weight on sulfate resistance was studied. Mortar strength tests up to 1 year showed that for cement containing no $CaCl_2$, the longer the initial curing period in water, the lower was the sulfate resistance, but no influence of the curing period in water was observed when $CaCl_2$ was added; sulfate resistance increased with increase in the initial curing period in the moist closet, especially when $CaCl_2$ was added. The increase in sulfate resistance due to the addition of $CaCl_2$ seems to be related to the acceleration of the formation of calcium sulfoaluminate and chloroaluminate at earlier ages of hydration.

### Experiments on the effect of calcium chloride on corrosion of steel reinforcement in concrete

Nobue Fukuda and Hiroshi Awata, *Semento Gijutsu Nenpo*, V. 10, 1956, pp. 188-193

Ceramic Abstracts
Sept. 1958 (Y. Suzukawa)

The effect of the addition of 2 percent by weight of $CaCl_2$ to cement on the corrosion of a mild steel plate of 50 x 30 x 3 mm embedded in mortar specimens of 20 x 20 x 20 cm (water:cement:sand ratio = 1:2:5) was tested up to 1 year. Weight loss due to corrosion was about 0.76 mg per sq cm during moist curing for 6 months. No further weight loss was observed in specimens cured up to 1 year. Various electrochemical measurements made on a mild steel plate in mortars prepared from cement containing 0 to 5 percent by weight of $CaCl_2$ showed that the layer of $CaCO_3$ formed on the surface has a protective action against corrosion. It was confirmed that $CaCl_2$ must not be used when there is a stray current in the place where the concrete is placed.

### Corrosion of steel in reinforced concrete (in French)

M. J. Brocard, Supplement to *Annales de L'Institut Technique du Batiment et des Travaux Publics* (Paris), No. 126, June 1958, Series Béton, Béton Armé (47), pp. 756-772

Permeability of concrete to air seems to have a not very marked effect on steel corrosion for usual percentages of cement, since the coefficients of permeability vary slightly when concrete is proportioned to give a "full" mix with a minimum of voids. For very low coefficients of permeability, mixes markedly richer than normal are required.

The rate of corrosion decreases in proportion to the thickness of concrete over steel reinforcement. Beyond 1.9 in., it becomes negligible; this thickness should be considered as a minimum for concrete exposed in an aggressive medium.

The effect of cracks on the steel corrosion depends essentially upon the width of cracks. Tests take into account the width of the cracks at the level of the reinforcement. For widths of less than 0.015 in., cracks do not appear to have an appreciable effect. Above this figure, the rate of corrosion increases rapidly and may easily increase to as much as ten times the initial rate.

## Pavements

### Experiments with protective coatings and with methods of repairing a scaled concrete road (in Swedish)

Nils Odemark and Sven Engman, *Report* 31, State Road Institute (Stockholm), 1957, 28 pp.

Highway Research Abstracts
July 1958

The tests were carried out on a pavement built in 1947 of concrete without entrained air. As far as is known, salt has never been used on this road. The road had no scalings until in 1953, when scalings occurred on a large part of the surface. The tests were of two different kinds: (a) tests of different types of surface treatments to prevent further scaling, and (b) experiments with different methods of repairing scaled spots.

Experiments with treating the concrete surface with various products to prevent further scaling included: (1) concentrated water-glass, (2) diluted water-glass and calcium chloride, (3) a mixture of diesel fuel and white spirit in proportions 1:3—3 percent stearic acid was added, (4) a water-repelling silicone product, and (5) a plastic product.

Experiments with different methods of repairing scaled spots were: (1) sandblasting, treatment with hydrochloric acid, washing with water and repairing with cement mortar laid by means of a trowel; (2) sandblasting and repairing by spraying cement mortar; (3) repairing with cement mortar to which had been added an emulsion of high-polymeric substance (repairing by means of a trowel,

repairing by means of spraying); and (4) repairing with cement mortar to which had been added an emulsion of a high-polymeric substance (repairing by means of a trowel, repairing by means of spraying).

Results of the last inspection in 1957, when the repair work was 3 years old, were used for a final judgment of test results.

Treatments were made both on undamaged and on partly scaled slabs and the development of new scaling was observed. It was found, in comparison with untreated slabs, that treatments of the kind here tested have no noticeable effect on the development of scaling.

Irrespective of method of repair, all patches were intact, unless minor damages at the edges are taken into consideration. It seems reasonable to conclude that good patching results mainly depend on a good cleaning of the surface which was here done by sandblasting. The result applies to pavements on which salt has never been used. The institute has lately made freezing and thawing tests on small concrete cubes, submerged in solutions of salt, and also, patching tests of a similar kind as those mentioned above. The laboratory tests on the cubes showed that admixture of a high-polymeric emulsion (products mainly consisting of polyvinyl acetate) improves the resistance of concrete to the combined action of salt and frost. The results of the further patching tests show that the use of admixture to the mortar is of advantage to improve adhesion to old concrete and, also, to make a mortar specially suitable to repair shallow scalings.

## Precast Concrete

### Fallacies in the current percent of total absorption method for determining and limiting the moisture content of concrete block

CARL A. MENZEL, *Proceedings*, ASTM, V. 57, 1957, pp. 1057-1071; discussion pp. 1072-1076

AUTHOR'S SUMMARY

The long-standing test for moisture content described in ASTM C 140-56 was intended to limit the permissible moisture content of concrete block. However, author feels it is inadequate since it fails to give a significant and realistic indication of the moisture condition of the concrete such as is needed to show whether or not block are at the desired degree of dryness to reduce drying shrinkage and cracking in the wall.

This paper presents data showing that the moisture held by air-dry concrete at equilibrium with a given relative humidity is influenced by many factors and is not significantly related to the total absorbed water in a saturated block. Hence, the current "percent of total absorption method" should be replaced by the "relative humidity method" which provides a reliable indication in about 20 to 60 min of the state of dryness and potentiality for drying shrinkage of concrete block in subsequent exposure to atmospheric conditions.

### First precast concrete curtain wall

*Architectural Record*, V. 124, No. 3, Sept. 1958, pp. 189-196

There have been a large variety of curtain walls for office buildings. This 15-story bank building used curtain wall composed of 2½ in. thick, precast faceted units, 5½ ft wide and 6 ft high, fastened to the structural frame.

## Prestressed Concrete

### Proceedings of the symposium on prestressed concrete and composite beams (in English and German)

Science Council of Japan, Architectural Institute of Japan, Japan Society of Civil Engineers (joint publishers), Tokyo, Nov. 1955, 97 pp.

The proceedings include papers on Japanese prestressing steels, railway bridges, railway ties, and research papers on bond, fire tests, corner stresses in prestressed rigid frames, shear connectors for composite beams, vibration characteristics and design and construction of prestressed earthquake resistant test structures.

### Behavior of prestressed concrete beams at transfer

A. C. SCORDELIS, T. Y. LIN, and H. R. MAY, *Structures and Materials Research Report*, University of California, Division of Civil Engineering, Series 100, Issue 3, Oct. 1957; available at $1.82 from the State of California Printing Division, Document Section, Sacramento 14, Calif.

The behavior of prestressed concrete beams at transfer under varying amounts of prestressed external moment acting to increase the eccentricity of prestress is discussed. Theoretical formulas are developed for the cracking and ultimate strengths at transfer

for rectangular and tee-shaped cross sections. Results of tests on 14 full-scale beams of various cross-sections are given. Test results generally verified theory, and both indicate that the present practice which bases the design at transfer on a fixed value of allowable tensile stress yields a widely varying factor of safety.

### Fire test of prestressed concrete slabs

KUNIO KAWAGOE, *Proceedings*, Symposium on Prestressed Concrete and Composite Beams, Tokyo, Sept. 4, 1954 (published Nov. 1955) pp. 27-31

Results of fire tests on beams, channel type slabs and plate-like slab members of prestressed concrete for building structural use are given. Describes test procedures and specimens used in this series of tests.

### Developments in prestressed concrete —a review prepared by the Prestressed Concrete Development Committee

*Proceedings*, Institution of Civil Engineers (London), V. 8 (Session 1957-58), Nov. 1957, pp. 293-322
Reviewed by ARON L. MIRSKY

A critical survey and review of developments since 1949 in materials, design fundamentals, techniques, experimental work; bridges, buildings, tanks, roads and runways, and other applications. While a general reference to the committee's Bibliography on Prestressed Concrete is made, keyed citations are unfortunately lacking.

### Study on vibration characteristics of prestressed concrete beam

YASUJI TAHARA, *Proceedings*, Symposium on Prestressed Concrete and Composite Beams, Tokyo, Sept. 4, 1954 (published Nov. 1955) pp. 45-52
AUTHOR'S SUMMARY

In the study of properties of a prestressed concrete beam subjected to vibration, the effect of hysteresis which is observed in the stress-strain diagram of concrete of such a case, should not be ignored. Such hysteresis appears under the condition that concrete itself is constantly in state of compression while vibration stress is being applied. This sort of effect is particularly noticeable at the initial stage of vibration but diminishes gradually, and after a certain given number of vibrations it becomes almost negligible. Thus the beam might lose a part of its strength finally as the result of the secondary loss of tension in piano wire, due to the total effect of hysteresis accumulated before the number of vibrations given.

Despite this interesting theoretical result, the numerical result obtained from some experiments shows that the influence is very small for the beam as usual in comparison with the effect of creep or plastic-flow caused by the loading mass on the beam, and is not significant in the practical design of a beam.

### Studies on the knee part of the prestressed concrete rigid frame structure

YASUO KONDO, *Proceedings*, Symposium on Prestressed Concrete and Composite Beams, Tokyo, Sept. 4, 1954 (published Nov. 1955) pp. 53-58
AUTHOR'S SUMMARY

Prestressed concrete structures now in Japan are used mainly as bridge structures of, in most cases, simple beam or slab constructions, and there are very few examples of statically indeterminate structures.

Reinforced concrete and also prestressed concrete structures are used most advantageously as Gerber beams, continuous beams or rigid frame structures. Rigid frame structures are most suitable in Japan where frequent earthquake takes place. Object of this test was to obtain further information of statistical behaviors of prestressed concrete rigid frame structures under loading. Three models of rigid frame structures of different types were used, and stresses distribution in the knee part was measured.

## Properties of Concrete

### Influence of age on the ball hammer testing of concrete with dense texture (Kugelschlagprüfung von Beton mit dichtem Gefüge. Einfluss de Prüfalters)

KURT GAEDE, *Bulletin* 128, Deutscher Ausschuss für Stahlbeton (Berlin), 1957, 18 pp., 6 DM
Reviewed by FRITZ KRAMRISCH

The report describes tests and evaluates test results regarding the influence of age on the ball hammer testing of stone concrete in order to determine its compressive strength. These tests have shown that hardening of the surface progresses faster than hardening of the interior and that, therefore, equal diameters of impressions, obtained on specimens of different age, indicate different compressive strengths of the specimens. The explanation for this behavior is sought in the faster

hardening of the air exposed surface. This characteristic affects not only the method that measures the diameter of the ball impression, but also the instruments that use the rebound as a measure of the compressive strength, because the plastic, gel-like character of young concrete can absorb more energy than the more elastic, crystalline character of older concrete that results in a greater rebound.

Test results are given and formulae are developed to adjust readings of impression diameters and rebounds measured on concrete having an age different from 28 days to the standard reading of 28-day concrete.

## Evaluation of hardened concrete

W. S. Butcher, *Journal*, Institution of Engineers (Australia) (Sidney), V. 30, No. 6, June 1958, pp. 185-189

Reviews methods available to determine strength, quality, condition, and other properties of concrete in a structure.

Tests discussed are: chemical analysis, core tests, dynamic tests, tests for water and air content, petrographic analysis, gamma and x-ray examination, and physical characteristics of the concrete in place. Despite the number of tests available, author cautions care in interpretation of results with due regard to the limitations of the tests so as not to read into the results more than is justified.

## Factors affecting measurement of compressive strength of insulating concretes

Morton Sherman, *Proceedings*, ASTM, V. 57, 1957, pp. 1096-1109; discussion pp. 1110-1121

Author's Summary

For lack of experimental evidence to the contrary, testing laboratories and design engineers have found it necessary in the past to apply to lightweight insulating concretes the techniques, limits, and standard relationships that are proper for dense aggregate concretes. Some of these assumptions now appear to be erroneous when applied to insulating concretes.

Data presented in this study indicate that:

1. The effect of loading rate on the measured strength does not appear to be as critical as a variation in the dry density of the concrete under test.

2. Abnormal specimen height at casting can facilitate the formation of density gradients in the concrete specimen to be tested.

3. Without abnormal specimen heights, the height-width ratio of a test specimen can vary from 0.5 to 2.0 without seriously affecting the measured strength.

4. Improper curing can decrease the measured compressive strength.

A method of testing is suggested which utilizes 2-in. oven dry cubes.

## Wear tests on concrete using the German standard method of test and machine

James L. Sawyer, *Proceedings*, ASTM, V. 57, 1957, pp. 1143-1153

The Germans have developed a machine and method for abrasion test (DIN 51951) which subjects the test specimen to wear due to forces that are similar to those experienced by floor surface in service. This paper reports work with the German standard machine using a slightly modified test method.

The test is sensitive to differences in mix and curing procedures. Results indicate that adequate curing is essential for good wear resistance. The increased wear due to surface moisture indicates that the expected moisture condition of the floor surface, when in service, must be considered in designing a suitable floor.

## Freezing-and-thawing resistance of concrete as affected by the method of test

H. L. Flack, *Proceedings*, ASTM, V. 57, 1957, pp. 1077-1095

Author's Summary

The data presented are a compilation of test results of several investigations performed by the Bureau of Reclamation to determine the factors which may influence results of laboratory freezing-and-thawing tests. Several combinations of freezing-and-thawing periods including four tentative ASTM methods were used to test the effect of initial cures, degree of saturation, and specimen size and shape. In addition, the time-temperature relationships established for different locations in an 18-in. cylindrical specimen to simulate the relative effect on structural concrete are shown for various patterns of freezing and thawing.

### Influence of clayey additions on the properties of mortars and concretes

NICOLAE MIHAIL, *Bulletin de l'Institut Politehnic Bucuresti* (Bucharest), V. 19, No. 1-2, 1957, pp. 135-140

CERAMIC ABSTRACTS
Sept. 1958 (M. Hartenheim)

The influence exerted depends largely on the dispersion of the clayey substance in the mass. If the clay is present, e.g., in the form of a thin coating, it has an unfavorable effect on the adhesion of the aggregate grains and on the properties of the mortar or concrete, as this coating prevents the binding of rocks and cement. If, however, the clayey substance is finely distributed in the mass, it improves the plasticity.

### Measuring the rate of hardening of concrete by bond pullout pins

T. M. KELLY and D. E. BRYANT, *Proceedings*, ASTM, V. 57, 1957, pp. 1029-1040; discussion pp. 1041-1042

AUTHORS' SUMMARY

The method consists of the determination of the rate of development of bond strength. Stainless steel pins were embedded vertically in a concrete specimen immediately after the concrete was mixed. Individual pins were subsequently pulled out at increasing time intervals and the load at bond failure measured in each case. Rate-of-hardening curves were obtained by plotting bond strengths against time. Curves are presented which illustrate the effects of variations in concrete mixes and storage temperature upon rate of hardening.

### Effect of the properties of coarse aggregate on the strength and cement requirement of precast concrete (in Russian)

S. S. GORDON, *Beton i Zhelezobeton* (Moscow), June 1958, pp. 215-218

Reviewed by D. WATSTEIN

The effect of the properties of several crushed stones and gravels on the strength of concrete was recently investigated. It was concluded that the rock used as coarse aggregate must have a compressive strength of 1.5 to 2 times the design strength of the concrete. With other conditions being equal, the cement factor could be varied 20 to 40 percent in obtaining a concrete of a given strength, while for a given cement factor the compressive strength varied up to 50 percent.

Bond between the aggregate and the cement paste was found to be better for stone having a fine grain or fine crystalline structure than for minerals consisting of coarse grains. A hypothesis was advanced that this bond is also dependent upon the degree of similarity between the mineralogical nature of the crushed stone and that of the cement paste. Thus, dense dolomite and limestone may prove to be superior to crushed granite of the same strength in view of the greater similarity of dolomite and calcite to hardened cement paste. This hypothesis is offered only with respect to concrete mixes which are not subject to severe frost action.

## General

### Civil engineering in the service of the Brussels Exposition (Le génie civil au service de l'Exposition de Bruxelles)

GUSTAVE WILLEMS, *Memoires*, Société des Ingénieurs Civils de France (Paris), V. 111, No. 1, Jan.-Feb. 1958, pp. 48-72

Reviewed by ARON L. MIRSKY

First part of paper deals with traffic—European, Belgian, and Brussels—and describes briefly some of the new facilities (tunnels, viaduct) constructed to facilitate traffic to the exposition. Second part describes very briefly a few of the bold and imaginative exposition structures.

### Works of Pier Luigi Nervi, The

Preface by PIER LUIGI NERVI; introduction by ERNESTO N. ROGERS; explanatory notes to illustrations by JURGEN JOEDICKE; Translation by ERNST PRIEFERT; Frederick A. Praeger, Inc., New York, 1957, 142 pp., $10

Only rarely can the printed page capture the beauty and "simplicity" of the structures designed by Nervi. This book has accomplished that in an outstanding presentation with all of Nervi's completed works and some of his proposed designs dramatically illustrated and described in detail. The book illustrates Nervi's methods of construction, particularly in the development of prefabricated materials. Reinforced concrete construction, particularly favored by Nervi, has been treated in great detail including his "ferro-cemento." A great number of explanatory construction sketches and detailed photographic treatments supported by short text, concentrating on the basic elements involved, are included.

# ACI News Letter

Vol. 30, No. 6 JOURNAL of the AMERICAN CONCRETE INSTITUTE December 1958

Los Angeles freeway network

# ACI Book of Standards

1958 Edition

## Thirteen ACI Standards in One Book

A collection of current ACI standards, recommended practices, and specifications. Your guide through a maze of construction problems.

**$4.00**

To ACI Members: **$2.00**

- **Test Procedure to Determine Relative Bond Value of Reinforcing Bars** (ACI 208-58)
- **Recommended Practice for Evaluation of Compression Test Results of Field Concrete** (ACI 214-57)
- **Building Code Requirements for Reinforced Concrete** (ACI 318-56)
- **Design of Concrete Pavements** (ACI 325-58)
- **Reinforced Concrete Chimneys** (ACI 505-54)
- **Winter Concreting** (ACI 604-56)
- **Selecting Proportions for Concrete** (ACI 613-54)
- **Measuring, Mixing and Placing Concrete** (ACI 614-42)
- **Application of Portland Cement Paint** (ACI 616-49)
- **Concrete Pavements and Concrete Bases** (ACI 617-58)
- **Precast Concrete Floor and Roof Units** (ACI 711-58)
- **Construction of Concrete Farm Silos** (ACI 714-46)
- **Application of Mortar by Pneumatic Pressure** (ACI 805-51)

AMERICAN CONCRETE INSTITUTE

**PUBLICATIONS**

**P.O. Box 4754, Redford Station Detroit 19, Mich.**

# ACI

News Letter Contents

December 1958

On the Cover—Looking easterly along Hollywood Freeway toward Los Angeles Civic Center. Fabulous freeway network unites the city with various outlying communities. More than 250 miles of freeways lead into the heart of Los Angeles, site for the ACI '59 Annual Meeting scheduled for February 23-26.

Dues Increase..... 3

1959 Annual ACI Convention, Los Angeles 7

Detroit Regional Meeting Review.....13
- Design Session.....13
- Construction Session.....15
- Building Code Session.....18
- Materials Session.....23
- ACI Building Session.....24
- Dedication Ceremonies.....26

Positions and Projects.....33

Honor Roll.....36

New Members.....37

Looking Ahead.....38

List of Advertisers.....40

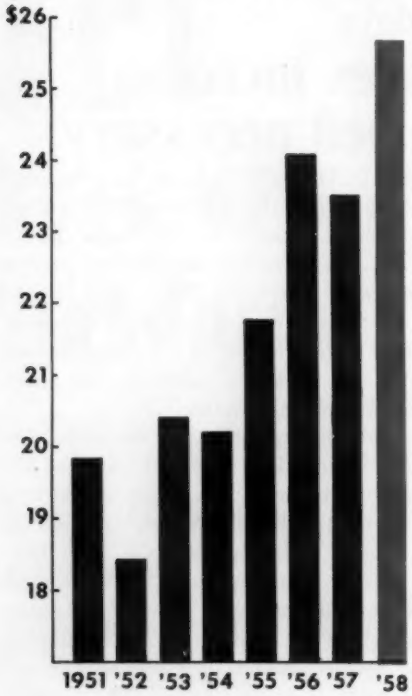
# To balance the budget

# dues increase deemed necessary

A YEAR'S STUDY of the Institute's financial status by a special committee of the Board of Direction resulted in a petition to the Board (see p. 5) on Oct. 28, 1958, to revise the Bylaws to provide for an increase in member dues. ACI Bylaws require that the petition be presented to the membership at the 1959 convention in Los Angeles and, if approved, passed to letter ballot of the entire voting membership shortly thereafter.

ACI dues have been maintained at the same level since 1951. In the interim the world—and ACI—have seen many changes; the most important to an organization on a fixed per capita income from members is that of creeping inflation with constantly rising costs. The chart on p. 4 shows the sharp upward trend in the last 8 years in per member cost of operating the Institute. Each member is now getting services which cost almost $26 per year, for which he is paying only $15. The increase in operating cost has accelerated in the past 3 years; 1958 expenditures will be almost $6 per member more than in 1951. Expenses are now converging on income with the threat that within the next year a deficit will result unless income is increased or services are curtailed. ACI has a modest reserve, but its relation to annual operating expenses is annually becoming more unfavorable; that is, percentagewise it is decreasing and receding from the recommended level of 1 year's operating expenses generally agreed to be the minimum desirable for stable operation of organizations like ACI.

After studying the special committee report the Board of Direction agreed that curtailment of services, or even maintenance of the *status quo*, is not the desire of the majority of members or the solution to the impending financial pinch. Rather, because of the backlog of accepted papers awaiting JOURNAL publication, the necessity and desirability of preparing and publishing a new index of ACI




**EXPENDITURES per member per year exclusive of publications purchased for resale. The trend of the past 8 years is sharply upward. Each member is now receiving services which cost almost $26 per year. The accelerated increase in operating costs over the past 3 years indicates that 1958 expenditures will be up almost $6 per member over 1951**

literature, and future work on new technical manuals, the consensus was for *more*, not less, service to members.

Obviously more publications and their development cost money. Compare the expenditure for the JOURNAL alone of $80,000 in 1958 with slightly over $23,000 in 1951. Steeply rising costs of printing and expanded issues of the JOURNAL account for the almost four-fold increase in this one expense item. The average direct printing cost per JOURNAL page has gone from $16.92 in 1951 to $40 in 1958. In addition to higher unit costs, the volume of literature sent to members has increased materially. JOURNAL V. 22 (*Proceedings* V. 47) ending in June 1951 had 872 pp.; JOURNAL V. 29 (*Proceedings* V. 54) ending with Part 2, December 1958 will run at least 1400 pp.

Increasing membership and services necessarily mean more staff, more equipment, and more working space. While the new ACI headquarters building has more than twice the area of the old office, the unit cost is materially less due to the generosity of members and friends of the Institute in contributing to the building fund which substantially paid for the new facilities.

So far, increasing costs of serving ACI members, over and above their dues, have been absorbed through increased revenues from bound volumes, reprints and special publications, and JOURNAL subscriptions. Prices of all these were increased in 1958. An increase in member dues has been deferred as long as possible but is now necessary to assure a continuing increase and growth in services worthy of a virile technical society while maintaining a level of financial stability essential to attainment of long-term ACI goals.

The need for greater member service exists; the raw material for developing it is available. A favorable member vote on the proposed dues increase will provide the financial means to accomplish it.

## Proposed Changes in ACI Bylaws

Notice is hereby given to the membership of the American Concrete Institute that at the Institute's 55th Annual Convention, Los Angeles, Feb. 23-26, 1959, consideration will be given to petition for Bylaws revision.

The undersigned members of the American Concrete Institute hereby petition the Board of Direction to submit to the membership for adoption, in accordance with Article VI of the Bylaws, the following proposed changes in the Bylaws. (The left-hand column repeats the present wording of the Bylaws; the right-hand column the proposed rewording.—EDITOR)

### PRESENT

### ARTICLE IV—DUES

*Section 1.* Dues shall be payable in advance on the first day of the month of notification of the Member applicant of his election by the Board of Direction, and annually thereafter, as follows: Contributing Members, $100.00; Corporation Members, $50.00; Members (individuals) in North America* and U. S. possessions, $15.00; Members elsewhere, $12.00; Junior Members, $7.50; Student Members, $5.00. Corporation Members outside North America may upon application be granted a 20 percent reduction in dues. Any individual Member may be admitted to Life Membership upon payment of a sum determined by the Executive Committee based on 90 percent of the membership dues as established at the time of application, credited with 3 percent interest compounded annually for the applicant's life expectancy as arrived at from the American Experience Table of Mortality.

### PROPOSED

### ARTICLE IV—DUES

*Section 1.* Dues shall be payable in advance on the first day of the month of notification of the Member applicant of his election by the Board of Direction, and annually thereafter, as follows: Contributing Members, $135.00; Corporation Members, $65.00; Members (individuals) in North America* and U. S. possessions, $20.00; Members elsewhere, $16.00; Junior Members, $10.00; Student Members, $5.00. Corporation Members outside North America may upon application be granted a 20 percent reduction in dues. Any individual Member may be admitted to Life Membership upon payment of a sum determined by the Executive Committee based on 90 percent of the membership dues as established at the time of application, credited with 3 percent interest compounded annually for the applicant's life expectancy as arrived at from the American Experience Table of Mortality.

Signed by:

WALTER H. PRICE
LEWIS H. TUTHILL
STANTON WALKER
BRUCE FOSTER
CLAYTON L. DAVIS
JOE W. KELLY
W. J. McCOY
C. D. WAILES, JR.
C. S. WHITNEY
BRYANT MATHER
FRANK KEREKES
ARSHAM AMIRIKIAN
H. J. GILKEY
RAYMOND C. REESE
CEDRIC WILLSON
PHIL M. FERGUSON
S. J. CHAMBERLIN
ROBERT C. JOHNSON
DOUGLAS McHENRY

*Includes all countries and territories in Central America and West Indies. Authorities: *Commercial Atlas of America*, Rand McNally & Co., Chicago; and *Survey Atlas of the World*, The Times, London.

**ATTENTION EXHIBITORS**

## 1959 ACI CONVENTION

**February 23-26—Los Angeles**

**The local committee for the 55th ACI Annual Convention in Los Angeles, Feb. 23-26, 1959, will sponsor an exhibit in conjunction with the convention.**

**Firms desiring to exhibit are requested to contact:**

**Glen Thomas**
**c/o Southern California Chapter**
**American Concrete Institute**
**Room 1058, 742 S. Hill St.**
**Los Angeles 14, Calif.**

# *Reinforced CONCRETE*

***Saves 14% on cost of luxury apartment building***

*PHOENIX TOWERS*

*Architect: Ralph C. Harris, A.I.A.*
*Phoenix, Arizona*

*Structural Engineer:*
*Miller Engineering Co.*
*Chicago, Illinois*

*General Contractor:*
*Del E. Webb*
*Construction Company*
*Phoenix, Arizona*

Newest and largest luxury co-operative apartment building in the Southwest is the handsome $3,000,000 fourteen-story Phoenix Towers. Architect Ralph C. Harris chose flat slab reinforced concrete construction which made it possible to cantilever for balconies and sunshades very economically. The absence of projecting beams into the ceilings of the rooms below in this type of construction eliminated the necessity for suspended plaster ceilings.

Architect Harris further points out that cost comparisons showed a 14% saving with reinforced concrete over other framing systems. Before you build, investigate this economical, flexible, and timesaving medium of construction. Compare—and you will design for reinforced concrete.

*Concrete Reinforcing Steel Institute*
*38 South Dearborn Street, Chicago 3, Illinois*

*California calling:*

# Los Angeles will welcome 55th annual ACI convention

*February 23-26, 1959*

SOUTHERN CALIFORNIA will welcome ACI members in late winter as the Institute meets for its 55th annual convention at the Statler Hilton in Los Angeles, February 23-26. Research, design, and construction reports will share the spotlight with official ACI committee work at sessions dealing with pavements, pipe, blast and radiation resistant structures, bridges, precast panels, concreting methods—to mention only a few of the varied topics slated for presentation.

An added attraction of the 55th Annual Convention is the student competition for concrete designs and applications being sponsored by the Southern California Chapter of ACI and the convention exhibits committee. The competition is open to all students in southern California colleges regularly enrolled in engineering or architecture, with four cash awards totaling $400 to be made. The awards will be for the best presentation of projects using portland cement concrete. Presentations may be in the form of scale models, architectural renderings, perspective drawings, or pictorial presentations. Entries will be on display at the convention, and prizes will be presented at the annual awards luncheon.

Technical committee "working sessions" and the annual business session will take up the first 2 days of the convention program on February 23 and 24. On Wednesday, February 25, the presentation of technical papers will begin with two sessions running concurrently through Thursday noon. The final research session is scheduled for Thursday afternoon. Present plans for the convention program (excluding technical committee meetings) are outlined below:

## TUESDAY AFTERNOON, FEBRUARY 24

### First General Session

**Proposed Revision of Institute Bylaws**

**Proposed Standards or Revision of Standards**
**Committee 605, Hot Weather Concreting**
**Committee 613, Proportioning of Lightweight Aggregate Structural Concrete**
**Committee 614, Measuring, Mixing, and Placing Concrete**

**Committee Report:**
**Committee 326, Shear and Diagonal Tension**

## WEDNESDAY MORNING, FEBRUARY 25

### Construction Session

**Erection of Precast Concrete**—J. L. Peterson, J. L. Peterson, Inc., Los Angeles

**Joinery of Precast Elements**—W. Howard Gerfen, consulting structural engineer, San Marino, Calif., and John R. Anderson, consulting structural engineer, Pasadena, Calif.

**Concrete Pavements on Cement-Treated Subgrades in California**—J. W. Trask, California Division of Highways, Marysville

**Qualification Plan for Ready-Mixed Concrete Plants**—C. E. Proudley, North Carolina State Highway Commission, Raleigh

**Cast-in-Place Concrete Pipe**—Louis Kristoff, Corps of Engineers, Sacramento, Calif.

### Special Problems in Concrete

**Properties of Shielding Concrete**—James Henrie, Atomics International, Canoga Park, Calif.

**Design and Performance of Special Purpose Blast Resistant Structures**—R. A. Williamson, Holmes and Narver, Inc., Los Angeles

**Influence of Grade of Steel on Blast Resistance of Reinforced Concrete Beams**—W. A. Shaw and J. R. Allgood, U. S. Naval Civil Engineering Laboratory, Port Hueneme, Calif.

**Surface Cooling of Mass Concrete as a Means of Controlling Cracking**—Roy W. Carlson, consulting engineer, Berkeley; Don Thayer, California Department of Water Resources, Sacramento

## WEDNESDAY AFTERNOON, FEBRUARY 25

### Design and Analysis Session

**Prestressed Concrete Lift Slab**—Edward K. Rice and Felix Kulka, T. Y. Lin and Associates, Van Nuys, Calif.

**Effect of Longitudinal Forces on a Portal Frame Supporting a Highway Bridge Deck**—Tung Au, Carnegie Institute of Technology, Pittsburgh, and Thomas D. Y. Fok, Youngstown University, Youngstown, Ohio

**Prestressed Concrete Shells for Grandstands and Roofs, Hippodrome at Caracas, Venezuela**—Henry M. Layne, structural engineer, Los Angeles, and T. Y. Lin, T. Y. Lin and Associates, Van Nuys, Calif.

**Laboratory Research on Pavements Continuously Reinforced with Deformed Bars**—J. L. Waling and M. J. Gutzwiller, Purdue University, Lafayette, Ind.

**Stresses and Deflections in Pavements Continuously Reinforced with Deformed Bars**—J. L. Waling and M. J. Gutzwiller, Purdue University, Lafayette, Ind.

## Concreting Materials and Methods

**Evaluation of Concrete and Mortar Mixes**—W. A. Cordon, Utah State University, Logan

**Shotcrete: Properties and Application to New Buildings**—Stanley Zynda, Gunite Contractors Association, Los Angeles

**Controlling Tremie Concrete with Admixtures**—J. Wayman Williams, Sika Chemical Corp., Passaic, N. J.

**Concrete and Concrete Materials for Glen Canyon Dam**—W. H. Price, L. P. Witte, and L. C. Porter, U. S. Bureau of Reclamation, Denver

# THURSDAY MORNING, FEBRUARY 26

## Design Research

**Effect of Bar Cutoff on Bond and Shear Strength of Reinforced Concrete Beams**—Phil M. Ferguson, University of Texas, Austin, and Farid N. Matloob, Mosul, Iraq

**Shear Strength of Two-Span Continuous Reinforced Concrete Beams**—Jose J. Rodriguez, Albert C. Bianchini, University of Illinois, Urbana; Ivan M. Viest, AASHO Road Test, Ottawa, Ill.; and Clyde E. Kesler, University of Illinois

**Torsional Resistance of Reinforced Concrete**—George C. Ernst, University of Nebraska, Lincoln

**Behavior of a Continuous Concrete Slab Prestressed in Two Directions**—A. C. Scordelis, T. Y. Lin, and R. Itaya, University of California, Berkeley

## Products and Precast Elements

**Precast and Prestressed Folded Plate Slabs**—H. H. Edwards, Leap Concrete, Inc., Lakeland, Fla.

**Plastic Forms for Architectural Concrete**—J. A. Hanson, Portland Cement Association, Chicago

**Exposed Aggregate Type of Decorative Panels**—C. D. Wailes, Jr., Wailes Precast Concrete Corp., Sun Valley, Calif.

**Glazed Face Concrete Block**—S. H. Westby, Portland Cement Association, Chicago

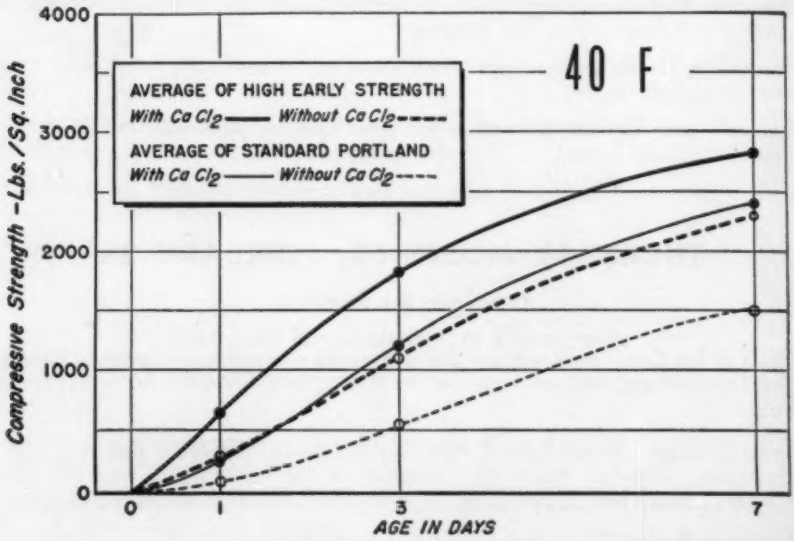
**Linings for Concrete Pipe and Structures under Sewage Conditions**—C. G. Munger, Amercoat Corp., South Gate, Calif.

# THURSDAY AFTERNOON, FEBRUARY 26

## Annual Research Session

# NO WINTER SLOW-DOWN HERE

## COLUMBIA CALCIUM CHLORIDE gives high early strength days faster




The American Concrete Institute now recommends the addition of calcium chloride for cold weather concreting. (ACI Standard "Recommended Practice for Winter Concreting.") These benefits have been proved, on jobsite after jobsite: initial and final set achieved up to three times faster, quicker early strength, greater ultimate strength. Finishing goes faster, forms can be pulled days earlier, contractor profit increases.

Let our Columbia Calcium Chloride engineers help you save money on your concrete work this season. For their assistance, just write today to Calcium Chloride Department at our Pittsburgh address, or to any of our fourteen conveniently located District Sales Offices.

Columbia Calcium Chloride is available in both High Test Flake (94-97% $CaCl_2$ content) and Regular Flake (77-80% $CaCl_2$). One 80 lb. bag of High Test delivers the same results as one 100 lb. bag of Regular. Both add easily at suppliers' plants or right at the jobsite.

COLUMBIA-SOUTHERN CHEMICAL CORPORATION

A Subsidiary of Pittsburgh Plate Glass Company

One Gateway Center, Pittsburgh 22, Pennsylvania

DISTRICT OFFICES Cincinnati, Charlotte, Chicago, Cleveland, Boston, New York, St. Louis, Minneapolis, New Orleans, Dallas, Houston, Pittsburgh, Philadelphia, San Francisco IN CANADA Standard Chemical Limited

**FLASH!!**

# Los Angeles Committee Plans Special Events For ACI '59 Annual Meeting

With the convention program proper concluding on Thursday, February 26, a trip to Disneyland is on tap for the following day in order that both delegates and wives may enjoy this popular southern California attraction. Bob Blair of Southwestern Portland Cement Co. and John Wise of the Disneyland staff are in charge of arrangements.

Sam Hobbs, local general chairman for the 1959 ACI convention announces that Henry M. Layne, Los Angeles consulting structural engineer, will act as chairman of the reception committee for the convention. Mr. Layne is a widely known engineer who has served as president of both the Los Angeles and California structural engineers associations. Other new local committee appointments include Charles F. Moran of the Portland Cement Association, chairman of the inspection trips committee, and William F. Norton of Ceco Steel Products Corp., treasurer. Mr. Norton is also a member of the over-all local planning committee.

Charles Pankow of Peter Kiewit Co. and Marvin J. Kudroff of Daniel, Mann, Johnson and Mendenhall are co-chairmen for the convention "social hour" on Wednesday night, February 25.

## Gala schedule for ladies

C. Taylor Test, Riverside Cement Co., serving as entertainment chairman of the local convention committee, has given "milady" prime consideration in special events planning.

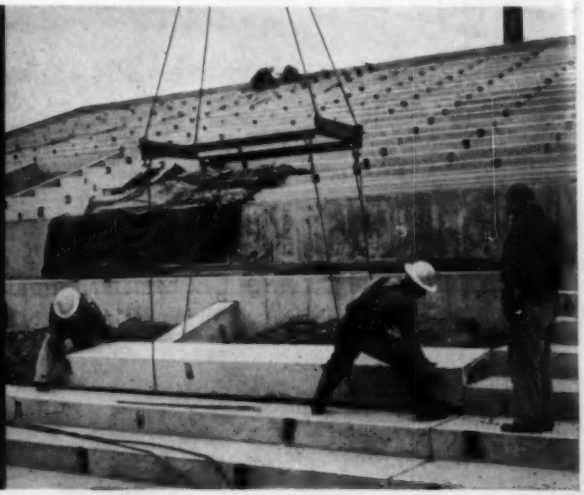
Entertainment scheduled solely for the ladies includes a get-acquainted buncheon and a fashion luncheon to be staged by a renowned Hollywood designer with studio models. Tickets to popular television shows and reservations for famous Los Angeles night clubs will also be available. There will be ample time, too, to take in the many attractions of the Los Angeles area—Hollywood Bowl, Catalina Island, Descanso Gardens, Huntington Library, Olvera Street, Knott's Berry Farm, Farmers Market, the beaches, mountains, or desert.

Certain to be one of the high spots for the ladies is the fashion luncheon scheduled for Thursday noon at the Beverly Hilton Hotel in Beverly Hills. Walter Plunkett, chief dress designer for Metro-Goldwyn-Mayer Studios and an Academy Award winner for design, will stage the fashion show. Mr. Plunkett will display his award-winning creations as well as current designs.

*Continued on p. 39*

NATIONAL BROADCASTING COMPANY STUDIOS in Hollywood—an example of the beauty and strength possible in architectural concrete. The use of this medium first gained favor on the West Coast, perhaps because of its high resistance to earthquake and also its flexibility and possibilities for modern function design. Tickets to popular television shows originating from this studio will be available to ACI convention-goers while they are in Los Angeles for the 55th Annual Convention, February 23-26,1959

# PRECAST SEAT BEAMS SPEED COMPLETION OF BALL PARK



Seat beams were anchored by welding plates on seat undersides to plates on grade beams. 1082 seat beams were required.

Weighing approximately 1800 lbs. apiece, they ranged in width from 2'8" to 3' and in length from 13' to 14'.

*Contractor for Structure:*
Troy B. Hood, Inc.
Spokane, Wash.

*Contractor for Excavation, Foundation and Tunnels:*
Henry George & Sons
Spokane, Wash.

*Manufacturer of Precast Units:*
Spokane Concrete Pipe Co.
Spokane, Wash.

*Ready-Mix Concrete:*
Central Pre-Mix Concrete Co.
Spokane, Wash.

*Architect and Engineer:*
Culler, Gale, Martell & Norrie
Spokane, Wash.

*Joint Owners:*
City of Spokane
Spokane County

• Structural work on the new Fairground Park baseball stadium for the "Spokane Indians" did not get underway until February, 1958, yet the job had to be completed by the season's opener on April 29th. This need for fast work, without sacrifice of economy or quality, led to the choice of precast concrete seat beams for this job.

Using Lehigh Early Strength Cement, the manufacturer turned out 44 seat beam sections every day, from 550 lineal ft. of forms. Without the use of High Early Strength concrete, at least double the quantity of forms would have been needed to meet the tight schedule.

This is typical of the advantages of Lehigh Early Strength Cement in modern concrete construction.

With modern construction methods and Lehigh Early Strength Cement, the manufacturer was able to complete casting of the beams for this 8500-seat baseball stadium in less than 5 weeks.

LEHIGH PORTLAND CEMENT COMPANY
ALLENTOWN, PA.

## *Record breaking attendance for three-day technical program at*

# DETROIT REGIONAL MEETING

EUROPEAN AND AMERICAN engineering developments in concrete formed the nucleus for the technical program presented at the 11th Regional Meeting of the American Concrete Institute, October 27-29, in Detroit. More people were registered than for any previous ACI Regional Meeting. The program featured five technical sessions and an inspection tour of the new ACI headquarters building which was formally dedicated October 29, crowning the meeting with a deep sense of membership accomplishment.

Official registration totaled 559, with Michigan providing the largest group of registrants, although members from 26 other states were in attendance. An international flavor definitely prevailed, with delegates from Canada, Belgium, Cuba, Germany, Japan, Mexico, Netherlands, Puerto Rico, and Sweden attending the sessions. Four Michigan universities were represented by 118 students.

## Design Session

ACI President Douglas McHenry served as moderator at the opening session on Monday, October 27, with five diversified design papers featured. Mr. McHenry is director of development, Portland Cement Association, Chicago.

### Russian Progress in Concrete Technology—J. D. Piper, vice-president for promotion, Portland Cement Association, Chicago, and Walter H. Price, head, Engineering Laboratories, U. S. Bureau of Reclamation, Denver

Past president Walter H. Price yielded the rostrum to coauthor J. D. Piper who presented an enlightening illustrated story of their recent trip to Russia. The authors were members of a six-man delegation invited to inspect concrete construction in Moscow and Leningrad from May 11 to 21, 1958, where they visited laboratories, design offices, construction sites, precasting plants, and housing developments.

They observed that prestressed concrete is used wherever possible in Russia. Precast concrete is being used almost exclusively for housing projects in the above mentioned cities. Roofs, floors, even heating units, are made of concrete.

Before starting a tour of each of the factories, design offices, and technical laboratories, the American delegation was introduced to officials who explained what they were trying to accomplish. Upon completion of the tour the delegates again returned to the executive offices where they were asked to comment on what they had seen and offer any possible constructive criticism or suggestions which might improve the present methods. These plants, working three shifts a day, 6 days a week, turn out a tremendous volume of precast concrete.

Colored slides afforded a broad picture of the 10-day tour depicting outstanding tourists' attractions such as the 32-story University of Moscow, the Kirov Stadium with seating capacity for 100,000, and the fabulous St. Basil subway. Built 100 ft below the ground, the subway is marble lined, with beautiful paintings, statues, and ornate decorations making the passageway comparable to a corridor of the finest art museum.

*Continued on p. 14*

**Concrete Space Structures — Relation Between Form and Structural Design —A. M. Haas, professor of concrete and concrete construction, Institute of Technology, Delft, Netherlands**

The amazingly rapid development of concrete design demands greater understanding between the architect and the structural engineer.

In the past, architectural physical planning was limited by the structural capabilities of the materials used. The stability of low tensile materials (like natural stone) required close, vertical, and huge supports aligned more or less on rectangular patterns. But now entirely new concepts of design have emerged, said Professor Haas. With the discovery of the possibility of vaulting for low tensile materials and the availability of concrete that can easily take the required form, supports were pushed much farther apart. Space started to take forms other than that of the rectangle.

**Prof. A. M. Haas, Institute of Technology, Delft, Netherlands, one of the featured speakers at the design session**

The recent introduction of materials strong in tension, i.e., steel and reinforced concrete, has pushed supports still farther apart, with units of space taking more horizontal proportions.

Any structure should be so designed that stability, stiffness, and strength result. The design should be an economical solution, but thus far we have developed in calculation such techniques that almost any fantastic architectural design may be made structurally possible. The consulting engineer hardly ever calls a halt to the conceptions or caprices of the architect. Complete understanding and cooperation between the architect and engineer would result in a more harmonious and balanced design, and probably a more economical one also.

Fully discussing the principles underlying the relation between form and structural design, Professor Haas cited the problem of column capitals and openings for flat slab design, the increasing demand from the architect's office to do away with column capitals, various types of shell structures, and the advantages of folded plate design.

**Strength of Concrete Under Combined Stresses—Boris Bresler and K. S. Pister, professor and associate professor of civil engineering, respectively, University of California, Berkeley**

Professor Bresler presented this paper which represents a final report of a project sponsored by the Reinforced Concrete Research Council at the University of California. The study reported had two principal objectives: (1) to verify the validity of a failure criterion for plain concrete based on a relationship between normal and shearing mean stresses; and (2) to apply the failure criterion to the determination of shearing strength of structural elements.

A criterion for failure of plain concrete subjected to combined stresses was established from tests of 65 tubular specimens tested to failure under various combinations of shearing and compressive stress. A procedure for determining the shearing strength of a special class of rectangular reinforced concrete beams without web reinforcement was developed. Excellent correlation was obtained between calculated and observed shearing strength of a limited group of beams. Full report appeared in the ACI JOURNAL, September 1958, p. 321.

**Analysis of Shear Test Data—I. M. Viest, bridge research engineer, AASHO Road Test, Ottawa, Ill.**

Illustrated talk by Dr. Viest presented an analytical study of the test data for a large variety of reinforced concrete structural members without shear reinforcement. The test data, accumulated over the past 10 years, indicated that the principal tensile stress is the primary cause of failures of beams without web reinforcement. His analysis based on the well known principal stress equation expresses the strength of beams in

**INTERNATIONAL CONCRETE EXPERTS . . . Professor Andre Paduart of Brussels University; ACI President Douglas McHenry; Robert F. Legget, National Research Council of Canada; and Professor Hubert Rusch, Institute of Technology, Munich**

diagonal tension as a function of the percentage of the longitudinal reinforcement, the beam dimensions, the strength of concrete, and the ratio of moment to shear.

A correlation of the test data on the basis of the theoretical analysis permits the derivation of a simplified empirical design equation covering both short and long beams. Dr. Viest also included a comparison of the design equation with the test data and with the current design values for beams without web reinforcement.

**Comparative Cost of Blast Resistant Design—Warren W. Yee, director of engineering, Structural and Civil Division, Smith, Hinchman, and Grylls Associates, Inc., Detroit**

Dr. Yee opened his talk with a discussion of nuclear blast resistant design criteria. For conventional design, he said, we are dealing with predetermined forces such as dead loads and live loads, and forces of nature, but for structures to withstand a nuclear blast the forces involved are variables of the type of explosion, the intensity, and geographical conditions between the explosion and the structure. With given design criteria, it becomes no more than a fundamental engineering problem to design an economical structure that will satisfy all the requirements.

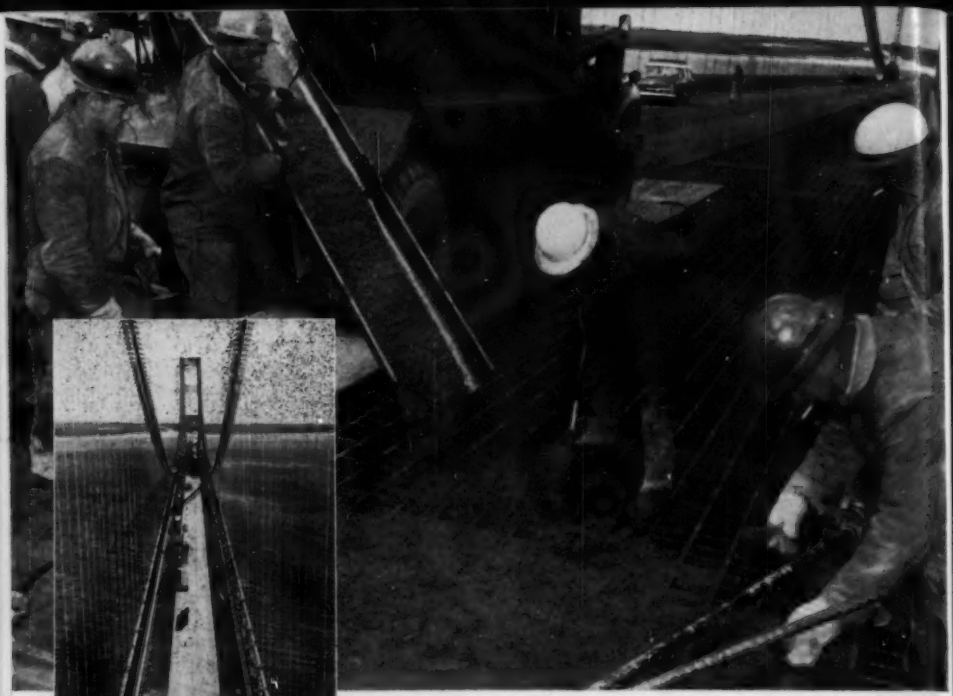
In discussing the structural requirements and framing schemes for a blast resistant structure, Dr. Yee called attention to the basic characteristics of a nuclear explosion and the resultant problems to be confronted.

Comparing the cost of blast resistant and conventional structures, he used a building constructed about 3 years ago, designed to resist all effects of a nuclear explosion of 20 kilotons intensity at a distance of ½ mile. Equivalent static loading for the structure is 150 psf on floors; 100 psf on the roof; and a lateral load of 150 psf on walls. All the loads are assumed to act in either direction. Cost for the building, excluding property and equipment was $15.86 per sq ft or $1.12 per cu ft. The increase in total construction cost of this blast resistant structure over that of a conventional structure was approximately 17 to 18 percent.

In conclusion, Dr. Yee pointed out that even with what appears to be staggering loading conditions, and radiation as well as thermal protection requirements of a blast resistant building, within reasonable limitations, one may still be designed and built at a cost less than 20 percent above that of a similar conventionally designed building.

## Construction Session

Shell roof construction problems, precast sandwich panels for shopping centers, design problems and service performance of precast panels, and the role of formwork in concrete construction were the topics of the construc-



**Mackinac Bridge, Michigan**

**Owner: Mackinac Bridge Authority**

**Consulting Engineer: Dr. D. B. Steinman**

**Associate Consultant: Glenn B. Woodruff**

**Substructure Contractor: Merritt-Chapman & Scott Corp.**

**Superstructure Contractor: American Bridge Div. U.S. Steel Corp.**

**POZZOLITH Ready-Mixed Concrete: Louis Garavaglia Co.**

# Mackinac Bridge... POZZOLITH* Employed in Lightweight Concrete in the Deck to Improve Workability and to Reduce Shrinkage

***POZZOLITH—Master Builders trademark for its time-tested water-reducing, air-entraining admixture for concrete.**

**THE MASTER BUILDERS CO.**
DIVISION OF AMERICAN-MARIETTA CO.

*General Offices:* Cleveland 3, Ohio • Toronto 9, Ontario • *Export:* New York 17, N Y
*Branch Offices In All Principal Cities* • *Cable Mastmethod, N. Y*

tion session, Monday afternoon, October 27. Ben Maibach, Jr. of Barton-Malow Co., Oak Park, Mich., presided as chairman.

### Concrete Shell Roof Construction Problems—O. G. Sharrar, general superintendent, O. W. Burke Co., Detroit

Mr. Sharrar reported that from the contractor's standpoint there were not many serious problems in this type of construction. He described the forming and concreting of a cylindrical shell roof for a water pumping station. Three 19 ft 5 in. vaults on each side of a passageway made up the roof which was 440 ft long and 3½ in. thick; the edge beams rested on columns 44 ft center to center. The barrel roof segments act as part of the beam.

Movable formwork, designed for 120 psf loading, was constructed for concreting the roof in sections. Supporting trusses were designed for a total of 17,500 lb. One truss was tested at that load and showed no deflection. The major consideration in concreting was to balance the placement so as to keep the load evenly distributed. The roof was cured for 12 days with water as required in the specifications.

### Precast Sandwich Panels for Shopping Center—R. C. Adams and R. D. Johnson, Marietta Concrete Corp., Marietta, Ohio

Presentation of this paper was made by R. C. Adams, who explained that his firm's standard precast panel is 5 in. thick, with concrete faces 1¾ in. thick and 1½ in. of rigid insulation between. An 8 x 10-ft panel was previously standard but now 8 x 20-ft panels are considered standard. Panels longer than 20 ft present certain problems in tolerances with regard to bowing, warping, and movement caused by temperature changes etc.

Advances in factory casting, trucking to the site, and mechanized erection have made the use of precast panels a valuable material in shopping center construction. Rapid closure of a structure allows the tenant to move in more quickly. Thinner walls provide more rentable space, and steel troweled interior surfaces of panels are easily painted. Exterior surfaces can be treated in a variety of ways with a variety of colors and textures.

Mr. Adams detailed the progress his firm has made in the field of precast sandwich panels and the adaptations required in plant, transportation, and erection brought about by the recent advances. His forecast is for greater use of precast panels, with an accompanying monetary saving, in the near future.

### Design Problems and Service Performance of Precast Concrete Wall Panels—Victor Leabu, assistant chief structural engineer, Giffels and Rossetti, Inc., Detroit

An engineer's point of view concerning precast concrete panels was presented by Victor Leabu, who offered some possible corrective measures to the problems of color variation, inefficient $U$ values, and warping of precast panels. Since concrete is the basic material of precast panels, the inherent problems affecting concrete (curing, moisture, shrinkage, and temperature differentials) affect panels as well.

Nonuniform coloring of natural concrete surfaces often results in unattractive walls. Special treatment of the outer surface to provide colors and textures is one solution, but the additional expense is not always warranted. Where natural concrete surfaces are used, greater control of ingredients is suggested. Matching of finished panels to minimize variations in a wall is another possibility. Uniform curing can reduce difference between panels. Avoiding the use of exposed steel parts (edge angles, connectors, etc.) will eliminate unsightly rust streaks.

Mr. Leabu feels that the responsibility for determining thermal properties of walls should be made definitely fixed, thus avoiding confusion which can affect heating costs. He pointed out the difference in the heat transmitting properties of sandwich walls through insulated portions and through rib supported areas, and that indicated values of $U$ should take this difference into account. He also suggests that the number of ribs and solid concrete stiffeners be kept to a minimum.

Warping of precast panels causes many cracks at ceiling, floor, and partition lines, but low slump concrete and use of curing compounds, water saturated covering, and steam curing will reduce this warping. Type of joint filler can also affect panel movement.

**Role of Formwork in Reinforced Concrete Construction—Harry Ellsberg, chief structural engineer, Giffels and Rossetti, Inc., Detroit**

Mr. Ellsberg, chairman of ACI Committee 622, Formwork for Concrete, sees a two-fold task facing the designer of forms for concrete construction. He must provide forms which are both economical and able to sustain the pressures of the flowing concrete. Since formwork cost often exceeds the cost of the concrete, the competitive position of concrete construction can depend on formwork design. Maximum reuse of forms will reduce costs, but careless and hasty form stripping and improper reshoring can have serious consequences.

The recommendation has been advanced that 6 x 6 lumber be used for reshoring instead of using the original 4 x 4's. With proper curing it has been noted that shoring of three stories is usually adequate for construction progress of one story per week. Also shoring should remain in place until a study is made of the loading due to work proceeding above.

There are two opposing viewpoints as to where the responsibility for formwork design should be placed. One group feels that all planning belongs in the drafting room, while the other argues that the contractor should be given only "end result" or performance requirements.

Future work of ACI Committee 622 will include determining recommendations for the formwork designers and a method for economically determining lateral pressures on forms. Several methods of measuring lateral pressure are being studied. At present the cost of a measuring device is prohibitive.

# DETROIT REGIONAL MEETING

## REGISTRATION RECORDS

**The increasing interest in Institute activities was evidenced at the Detroit regional meeting with official registration totaling 559. While the greatest contingent emanated from Michigan, 26 other states were represented at the meeting. Registrants from 9 countries were in attendance with Canada having 26 representatives present. Four Michigan universities were represented by 118 students.**

**Following are registration totals:**

| | | | |
|---|---|---|---|
| Michigan | 317 | Texas | 2 |
| Illinois | 68 | Alabama | 1 |
| Ohio | 44 | Florida | 1 |
| Canada | 26 | Kansas | 1 |
| New York | 26 | Kentucky | 1 |
| District of Columbia | 13 | Mississippi | 1 |
| Pennsylvania | 10 | New Mexico | 1 |
| California | 8 | Tennessee | 1 |
| Colorado | 5 | Virginia | 1 |
| Iowa | 5 | West Virginia | 1 |
| Massachusetts | 5 | Belgium | 1 |
| New Jersey | 5 | Cuba | 1 |
| Wisconsin | 5 | Germany | 1 |
| Indiana | 3 | Japan | 1 |
| Maryland | 3 | Mexico | 1 |
| Missouri | 3 | Netherlands | 1 |
| Georgia | 2 | Puerto Rico | 1 |
| Minnesota | 2 | Sweden | 1 |

## Building Code Session

Raymond C. Reese, consulting engineer, Toledo, Ohio, chairman of ACI Committee 318, presided at the Tuesday morning session, October 28, devoted to changing design practices, the influence of new materials and design

philosophy, and the relationship of these shifting ideas to impending changes in the ACI Building Code.

**Use of High Strength Steel in Reinforced Concrete—Georg Wastlund, professor and director of the Swedish Cement and Concrete Institute at the Royal Institute of Technology, Stockholm, Sweden**

Swedish high strength Kam steel and Austrian Tor steel were compared by Professor Wästlund, along with other types of high strength steels developed in Europe. He then considered the effect of development of high yield point steels on structural requirements with reference to reinforcement in three different realms: adequate safety against failure, limitation of crack formation, and sufficient rigidity (limitation of deflection). The fact that the relation between the ultimate load and the *reinforcement* is fairly well known makes it possible to increase the permissible stresses in bending approximately in proportion to the yield point. However, further research is in progress to eliminate the greater uncertainties existing when the theory of ultimate loads is applied to shear failures. Limiting considerations of cracking and deflection may make it impossible to take advantage of the higher allowable stresses anyway.

Professor Wästlund also discussed formulas for crack width and spacing, the bending properties of reinforcing steel, and compared Swedish, German, and American bending tests. He believes that American practice has proved the possibility of using deformed bars, thus dispensing with end hooks, insuring better bond, and reducing the size of cracks. European practice has benefited from these American developments, and has gone on to use higher strength steels which enable higher permissible stresses and more economical designs.

**Safety; Simple and Composite Bending—Hubert Rusch, professor of reinforced concrete and bridge building and Director of the Laboratory for Testing Building Materials and Structures, Institute of Technology, Munich, Germany**

According to Professor Rüsch, limit design is a much more suitable way for attaining uniform safety than is the system of permissible stresses. The load factor must cover the uncertainties menacing stability of a structure—uncertainties largely due to ignorance or lack of care. The common belief that the load factor should allow for a possible overload is not correct; failures occur at loading conditions which were included in the design. In choosing the actual load factor, the designer must take into account the consequences of possible failure; he must also realize that failures occur with decrease in loads; e.g., a highly prestressed beam may fail at the lower edge under compression when the moment value reaches a minimum.

**Raymond C. Reese, left, presided at the Building Code Session and Georg Wastlund, Cement and Concrete Institute, Stockholm, Sweden, who presented the session's opening paper**

Selected load factors must be larger if an *average* concrete strength is assumed than if some minimum concrete strength is assumed, based on statistical distribution of concrete strength test results. Load factors for live and dead loads should be the same only if design values assumed for dead and live loading have the same probability of occurrence.

Professor Rüsch also discussed his method of developing a stress block to be used for the study of simple and composite bending.

**What's Coming in ACI Building Code Revision?—Raymond C. Reese, consulting engineer, Toledo, Ohio**

Concrete research programs that began to accelerate some 10 years ago are now bear-

**PLAIN**
**1 : 2 : 4 Mix**

**Durability**
**Factor: 100**

**WITH AIR-ENTRAINING AGENT**
**1 : 2 : 4 Mix**

**Durability**
**Factor: 234**

**WITH PLACEWEL**
**1 : 2 : 4 Mix**

**Durability**
**Factor: 350**

# Now... from Johns-Manville...

## 3½ times better freeze-thaw resistance with Placewel,® the liquid water-reducer

The above photos show the results of 21 freeze-thaw cycles in a 5% calcium chloride solution. Under standard A.S.T.M. procedures, Placewel concrete had 3½ times greater resistance than plain concrete and was 1½ times better than straight air-entrained concrete.

*Liquid* Placewel contains air-entraining and dispersing-water reducing agents, *plus* a catalyst to step up hydrolysis and hydration of cement. By breaking up cement flocs, it releases water for lubrication which would not otherwise be available . . . requires less water in a mix without relying on air-entrainment alone.

*You get all the advantages of entrained air without its adverse effects on strength.*

Less than 3 oz. Placewel/sack portland cement will—

- increase strength 30% or more (after 28 days' curing)
- improve quality at no extra material cost
- increase workability and placeability
- reduce bleeding and segregation
- give controlled air entrainment
- reduce construction, maintenance costs

For complete technical assistance and the address of our representative nearest you, contact Johns-Manville, Box 14, New York 16, N.Y. In Canada, Port Credit, Ontario.

JOHNS-MANVILLE JM PRODUCTS

**JOHNS-MANVILLE**

ing fruit that demands further revisions in the ACI Building Code. Mr. Reese cited the work of private and public agencies in the United States, and the growing cooperation between ACI and the Comité Europeén du Béton.

Stronger steels and concretes available for construction emphasize the potential of ultimate strength design, which may move from the appendix to the main body of the Code in the next revision. Other topics being studied among pending Code revisions include: limitations of deflection and crack width for the slenderer flexural members being developed; establishment of overload factors for use in ultimate strength design; computation methods for diagonal tension and bond; flat slab and flat plate design provisions; footings; and eccentrically loaded columns.

When the next code revision appears, Mr. Reese said, the section on flexure will be divided into three parts: (1) all general requirements; (2) all requirements for working stress; and (3) all requirements for ultimate strength design. All requirements for specified types of structures will be accumulated in one place, all material on designs in another, and on details in another, permitting considerably easier reference.

**Status of Ultimate Strength Design—George Winter, professor and head, Department of Structural Engineering, Cornell University, Ithaca, N. Y.**

Professor Winter reported on some of the studies being made by Committee 318's subcommittee on ultimate strength design, of which he is chairman. Ultimate strength design, he said, permits one to exploit the real and proven superior strength obtainable with high strength steel, which results in saving about 30 percent of the total cost of installed steel. Deflections should be checked when $p$ exceeds $0.18\ f_c'/f_y$ to prevent the use of excessively shallow members.

Professor Winter confirmed Committee 318 agreement with ASTM proposals for a numerical designation system for reinforcing steels using the guaranteed minimum yield point in ksi as part of the bar identification, and proposed that bar size and quality symbols be rolled into the bar at regular intervals on the body of the bars.

The rectangular stress block as proposed by Whitney and others is being checked for adaptations to make it possible to include it in the next edition of the Code for ultimate strength computations.

**Beam and Creep Deflection—George E. Large, professor of structural engineering and chairman, Civil Engineering Department, Ohio State University, Columbus**

First summarizing some of the generally accepted basic principles of creep, Professor Large then pointed out that Sections 305(a) and 706(b) of the present ACI Building Code now recognize creep effects to some extent. Section 706(b) permits $2n$ to be used in transforming the area of the compression steel in flexural members, but he feels this does not go far enough, especially with regard to expressing the actual ratio of the compressive steel stress to the corresponding concrete stress.

Using Shank's creep method as a yardstick, Professor Large compared its values for stress in compression steel with those computed by other methods. He also presented a proposed revision of Section 706(b) covering allowable beam stresses and deflections with due recognition given to the effects of creep.

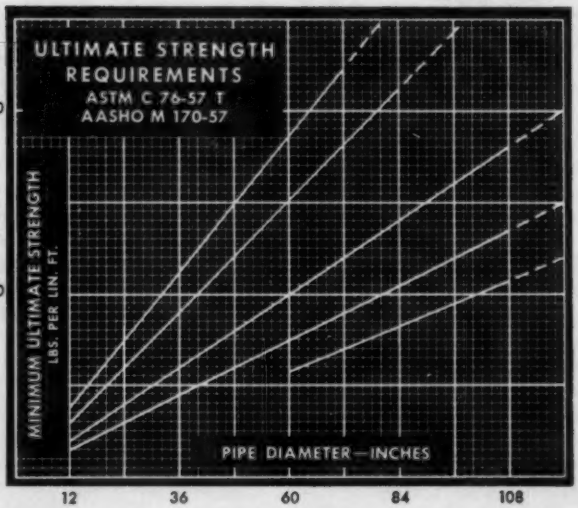
**Footings—Paul Rogers, consulting engineer, Chicago**

Footings do not as a rule play a major part in total construction cost, yet they support the entire structure, and their present low safety factor does not seem justified. Mr. Rogers advised that Code provisions be changed to require reinforcement in isolated footings to be selected for 100 percent of the moments. He also suggested that ultimate strength design provisions be carried into foundation design whenever that method is used for the rest of the structure.

Combined footings, raft and mat foundations, and similar supporting media will be studied by the newly formed ACI Committee 336, Combined Footings, under the leadership of Mr. Rogers. He does not expect that the results of that committee's work will be available for the next Code revision; hence he anticipates that Code provisions on those footings will be general, based on good design practice, perhaps warning of design by "experienced engineers only."

**COMPARE THESE CONCRETE PIPE STRENGTHS WITH ANY OTHER**

All classes of reinforced concrete pipe for greater strength and safety factor —as well as standard and extra strength non-reinforced concrete pipe— are available from American-Marietta Company to meet ASTM, AASHO and Federal Specifications.




**AMERICAN-MARIETTA COMPANY**
**CONCRETE PRODUCTS DIVISION**
GENERAL OFFICES:
AMERICAN-MARIETTA BUILDING
101 EAST ONTARIO STREET, CHICAGO 11, ILLINOIS, PHONE: WHITEHALL 4-5600

### Viewpoints on Flat Slabs—A Symposium

Four participants, M. P. Van Buren, partner, Di Stasio and Van Buren, New York; Alfred Zweig, senior structural engineer, Albert Kahn Associated Architects and Engineers, Inc., Detroit; Elihu Geer, professor and chairman, Department of Civil Engineering, University of Detroit; and Joseph C. Watts, Harley, Ellington and Day, Inc., Detroit, commented on the past, present, and future of flat slab design.

Mr. Van Buren, filling a program spot previously assigned to his partner, the late Joseph DiStasio, Sr., presented a comprehensive review of the history of Code provisions for flat slabs from 1917 to 1956. He remarked that specifications for shear reinforcement in flat slabs are being held in abeyance until current research is completed.

A number of revisions and additions to Chapter 10 (flat slabs) of the Code were suggested by Mr. Zweig, including: (1) relocation of the critical design section near the exterior columns using the elastic method; (2) limiting the empirical method to cases where the live load does not exceed three times the dead load; and (3) requiring alternate panel loading for elastic design for live loads exceeding three times the dead.

He also advocated permitting no reduction of analytically determined moments; relinquishing requirement for minimum moment of inertia for columns; increasing positive design moment for waffle slabs; and formulating shear design for flat slabs with shear heads.

Professor Geer suggested that the bond formula [Eq. (6)] of the Code does not apply to flat slabs, and proposed the following equation for bond in flat slabs, subject to empirical verification:

$$u = \frac{dMr/dr}{\sum ojd}$$

Using equations from Timoshenko's *Theory of Plates and Shells*, he presented a chart of $(dMr/dr)/Q$ values which can could be readily multiplied by shear $Q$ to obtain $dMr/dr$ for use in the bond equation.

Mr. Watts concluded the session in a lighter vein with his whimsical practicing engineer's view of the ACI Code. Sympathizing with the designer who must maintain "a decent regard for the laws of statics and the innate perversity of inanimate steel, concrete, timber, and soil conditions, and yet give the owner a champagne job for the price of beer," he took an optimistic view of improvements and simplifications in flat slab design procedures.

## Materials Session

The Materials Session, one of two concurrent sessions on Tuesday afternoon, October 28, heard discussion of heavy media separation of aggregates, finishes and colors for precast wall panels, efficiency of stationary tilting mixers, and lightweight concrete made with expanded slag aggregate. Paul F. Rice, Concrete Reinforcing Steel Institute, Chicago, presided as chairman.

### Heavy Media Separation of Aggregate —David A. Killins, secretary-treasurer, Killins Gravel Co., Ann Arbor, Mich.

Mr. Killins described heavy media separation equipment installed in a plant which was previously producing "borderline" aggregate, with respect to Michigan State Highway Department specifications. The installation made it possible to produce premium aggregate at 20 cents per ton additional cost.

Southeastern Michigan gravel deposits are glacial in origin and a mixture of sand and pebbles. A distribution of about 40 percent pebbles and 60 percent sand is considered a good deposit. Undesirable and deleterious materials vary among deposits but most are borderline in meeting highway department requirements. Coarse aggregate is undesirable if it is porous and absorbs water sufficient to burst the particles on freezing, or is a chert or clay material. Objectionable particles normally have a lower specific gravity than good material, lending themselves easily to heavy media separation.

The separation system described consisted of a revolving drum through which the coarse aggregate is channeled. A mixture of 40 percent ferrosilicon and 60 percent magnetite with water is the separating medium having a specific gravity of 2.58. The good material sinks to the bottom of the drum and is carried on lifters to the top and deposited on screens. The lighter, undesirable material is floated

over the lip of the drum. Undiluted medium is recycled through the system. The aggregate is washed and the diluted medium is passed through a magnetic separator where the metallic ingredients are removed for reuse. The "float" material is marketed for gravel driveways, septic fields and other drainage jobs, and also mixed with strippings as a road gravel.

**Special Finishes and Colors in Precast Concrete Wall Panels—Wilburn H. F. Saia, Precast Concrete Products, Royal Oak, Mich.**

The development of methods of manufacturing and erecting precast wall panels, and methods of applying finishes and colors has led to the acceptance of concrete as a wall finish material, according to Mr. Saia. Horizontal casting and factory curing have permitted treatments on concrete panels heretofore impractical. Among the varied treatments now used are pneumatically applied concrete to give various colors and textures, application of flaked mica for a sparkled effect, plastic finishes which give a porcelain enamel effect, exposed natural aggregate or particles of manufactured products, broom textured surfaces, and a finish of ceramic or glass mosaic tile which offer many color possibilities.

**Efficiency of Stationary Tilting Mixers —John Winkworth, Winkworth Fuel and Supply Co., Detroit**

Mr. Winkworth reported on a preliminary investigation, under the sponsorship of the ASTM task group on investigation of large mixers, to determine (1) the effect of mixing time on strength, cement distribution, slump and air content, and (2) to make a comparison between a 3- and 6-cu yd mixer. The tests were performed at two operating ready-mixed concrete installations.

Materials were representative of those used regularly at the plants and came from the same sources. Mix proportions were the same at both plants. Mixing time began after the mixer was fully charged. Samples were taken from front and rear of the mixer at 60, 90, 135, and 180 sec.

Slump tests revealed that at the end of 60 sec the slump difference in the 3-cu yd mixer was nominal, and no significant differences in appearance or workability could be observed. For the 6-cu yd mixer significant differences were present at both 60 and 90 sec. A revision of the procedure corrected this in subsequent trials. Unit weight determinations of the 3-cu yd mixer indicated satisfactory efficiency at the cycles investigated. An apparent tendency toward decrease in unit weight as mixing continued beyond 60 sec was indicated. Both mixers indicated excellent cement distribution. Soniscope tests were inconclusive since it was discovered that curing was not the same at both plants as had been assumed. As mixing time increased, and concrete became more workable, strengths of concrete from the 6-cu yd mixer became more nearly the same as those from the 3-cu yd mixer.

**Lightweight Concrete Made with Expanded Blast Furnace Slag Aggregate —D. W. Lewis, chief engineer, National Slag Association, Washington, D. C.**

Mr. Lewis reported on tests which provide data on both insulating and structural lightweight concrete. The tests showed that expanded slag concretes, in general, had excellent durability in the laboratory freezing and thawing tests. Structural mixes were unaffected by exposure up to 300 cycles of freezing in water and thawing in air. They also showed 28-day compressive strengths of 2000 to 4000 psi. Thermal conductivities varied directly with unit weight with $K$ values from 1.5 to 3.2 for unit weights from 60 to 103 lb per cu ft. This paper was published in its entirety in the November ACI JOURNAL, p. 619.

## ACI Building Session

William Krell, Portland Cement Association district structural engineer, Detroit, presided at the Tuesday afternoon session devoted primarily to reports on the design and construction of the ACI headquarters building. Mr. Krell provided the commentary for a color film showing precasting and erection of the ACI building's unique folded plate roof, then introduced architect Minoru Yamasaki, of Yamasaki, Leinweber and Associates, Birmingham, Mich. Commenting briefly on his experiences with planning

...consider concrete reinforced with

# WELDED WIRE FABRIC

Whenever construction calls for concrete, it usually pays to reinforce it with Welded Wire Fabric. Many contractors make extra sales to owners by carrying a few rolls of CF&I Clinton Welded Wire Fabric to the construction site. A short explanation of how Welded Wire Fabric substantially increases the life and appearance of concrete convinces many builders to choose reinforced concrete.

Concrete reinforced with Clinton Welded Wire Fabric has far greater strength than an unreinforced slab. It has higher resistance to the heaving, contraction and expansion caused by sudden temperature changes. If a crack should develop, the fabric holds it tightly in check, preventing moisture or earth from entering and expanding it.

Clinton Welded Wire Fabric is readily available, in all popular sizes and lengths in both the East and West. So, before you pour, consider reinforcing concrete with Clinton Welded Wire Fabric. The slight additional cost of reinforced concrete is more than offset by its longer life, more attractive appearance and minimum maintenance costs.

WHEN THEY ASK... "Is it Reinforced" SAY YES...WITH

**CLINTON**
**WELDED WIRE FABRIC**
THE COLORADO FUEL AND IRON CORPORATION 5694

THE COLORADO FUEL AND IRON CORPORATION—Albuquerque • Amarillo • Billings • Boise • Butte • Denver • El Paso Ft. Worth • Houston • Kansas City • Lincoln (Neb.) • Los Angeles • Oakland • Oklahoma City • Phoenix • Portland • Pueblo Salt Lake City • San Francisco • San Leandro • Seattle • Spokane • Wichita • WICKWIRE SPENCER STEEL DIVISION: Atlanta • Boston • Buffalo • Chicago • Detroit • New Orleans • New York • Philadelphia • CF&I OFFICES IN CANADA: Montreal • Toronto • CANADIAN REPRESENTATIVES AT: Calgary • Edmonton • Vancouver • Winnipeg

**MRS. HELEN KENNEDY, wife of the late Henry L. Kennedy, was guest of the Institute at the dedication luncheon. Past president Kennedy headed the ACI Building Committee, leading in the planning and fund raising activity for the headquarters building**

ACI headquarters, Mr. Yamasaki reemphasized his enthusiasm for precast concrete as a means to richness, silhouette, and form in architectural construction. He showed a number of pictures of his firm's work now in process, all designed since the ACI headquarters, and all using precast concrete.

John Strang, vice-president, Pulte-Strang, Inc., Ferndale, Mich., recalling some of the problems in construction for ACI, suggested that designers could cut concreting costs by planning for the use of standard size sheets of form material and standard sizes of lumber for formwork. He urged more designer attention to workable and presentable connections in precast concrete. He suggested use of textured finish for concrete work to reduce finishing problems.

C. S. Whitney, partner, Ammann and Whitney, New York and Milwaukee, spoke briefly of structural design for the Institute building; his remarks were published in the October JOURNAL, p. 427, in a paper entitled "Cantilevered Folded Plate Roofs ACI Headquarters." Papers by Messrs. Yamasaki and Strang also appeared in the October JOURNAL.

### Paduart reports on Brussels Fair design

Fortunate last-minute arrangements brought to the regional meeting program Professor André Paduart, University of Brussels, Brussels, Belgium, eminent European engineer and specialist in concrete. Professor Paduart, structural designer of the extraordinary Belgian Civil Engineering Pavilion at the Brussels International and Universal Exhibition of 1958, gave an abundantly illustrated account of design and construction of that noteworthy exhibit. The "spar," as it is sometimes called, is a concrete beam which cantilevers 250 ft over a giant map of Belgium, with visitors viewing the map from a concrete slab footbridge supported 17 ft above the ground by steel pipe hangers from the spar. The giant cantilever intersects a triangular shell, 95 ft on a side, near its base, and the whole is supported on only three footings.

Construction problems recounted by Professor Paduart included a winter construction schedule required because of the exhibitions opening deadline. Heated materials, paper and wood coverings, were necessary for December casting. All formwork was of 6-7 in. boards and joints between the boards appear in the finished concrete. Narrow strips of wood attached to the forms produced a clean line on concrete surface where each day's concreting stopped, and also kept fresh concrete from sliding along the sloping forms. Decentering in January was carefully planned so that no reversals of stress would occur in the beam; jacks were unloaded at varying rates in proportion to anticipated deflection.

## Dedication Ceremonies

Wednesday, October 29, was devoted to the official dedication of the new Institute headquarters building. Starting at 9 a.m. buses left regional meeting headquarters at regular intervals, conveying approximately 500 persons to the building site at 22400 West Seven Mile Road for a tour through the ultramodern ACI structure.

Following the tours a dedication luncheon was held in the ballroom of the Statler Hilton Hotel with Dr. A. Allan Bates, vice-president of the Portland Cement Association and chairman of the American Concrete Institute Building Committee, serving as toastmaster.

Dr. Bates introduced notable guests from the United States and abroad as well as the entire ACI Building Committee, the architect,

**Intricately decorated cake—a replica of the ACI headquarters building—is placed before Dr. A. Allan Bates, chairman of the ACI Building Committee. Seated to the right of Dr. Bates at the speakers' table is ACI vice-president Joe W. Kelly, J. Gardner Martin, general chairman of the regional meeting, and ACI past president C. S. Whitney. Note photo mural of ACI headquarters in background**

the structural engineer, and the contractor, all of whom played important roles in the construction of the ACI building.

Dr. Bates briefly discussed the architectural aspects of the unique headquarters structure which represents the end result of 10 years effort on the part of the late Henry L. Kennedy and the ACI Building Committee and then introduced ACI President Douglas McHenry, director of development, Portland Cement Association.

President McHenry provided an historical summary of ACI activities. From its inception in 1905 when it was known as the National Association of Cement Users, the aims of the organization were not materially different from the present aims of the American Concrete Institute. President McHenry commented on the manner in which ACI has kept pace with new developments and industrial growth, as well as the international scope of Institute activities. His closing remarks stressed that ACI progress during the past 50 years hurls a real challenge to present and future members.

## Legget presents world view of concrete

Robert F. Legget, director of building research, National Research Council of Canada, Ottawa, was guest speaker at the luncheon, tying his "Commentary on Concrete at a Time of Celebration" into the theme of the official dedication. Mr. Leggett's address dealt mainly with "where have we been with this wonderful material called concrete" and "where are we going?" Turning back the pages of history some 100 years, Mr. Legget briefed his audience on concrete in its earlier development and commented on a century of remarkable advance in its use, its versatility, and possibilities for combining beauty with utility. Mr. Legget discussed the wide use of concrete in other countries. His "birds-eye" view of the world touched New Zealand, Australia, China, India, Western Europe, and Scandinavia where much splendid concrete work is now in evidence. Concluding his commentary, Mr. Legget presented to President McHenry, in behalf of the Institute, a piece of Roman concrete, 2000 years old, as a token of good wishes from Canadian engineers and research workers, and as a constant reminder of our debt to the past, a symbolic token of all that has been done down through the ages in developing the concrete technology of today.

Ending the dedication ceremony in a "blaze of glory" all of the lights in the ballroom were dimmed and spotlights focused on a replica of the Institute headquarters building in the form of an intricately decorated cake.

*Continued on p. 28*

**LABYRINTH WATERSTOPS**

**A SOUND INVESTMENT FOR CONCRETE CONSTRUCTION!**

LABYRINTH AVAILABLE IN 2, 3 or 4 rib.

**ON YOUR CONSTRUCTION:**

**1.** Consider the investment in design, materials and labor (to mention a few).

**2.** Then consider how important safe, secure *watertight* concrete joints are.

**3.** Thorough watertightness *can* be secured by installing Labyrinth Waterstops – a dividend that makes the low initial cost of the product insignificant when compared to your total investment–and one that insures watertight concrete joints for years!

- Corrugated ribs grip concrete, insure an everlasting bond between joints.
- Finest polyvinyl plastic resists chemical action, aging, severe weather.
- Takes just seconds to nail to form . . . easy to cut and splice on location (prefabricated fittings available).
- There's a Water Seal product for every type of concrete work!

If your aim is to stop water seepage, *stop it effectively with Water Seals' Waterstops!*

**"See Us in SWEET'S"**

New Literature and Free Samples Sent on Request–Use Coupon Below

**WATER SEALS, *inc.***

**9 South Clinton Street, Chicago 6, Illinois**

Made in Canada for **J. E. Goodman Sales, Ltd. Toronto, Ontario**

**WATER SEALS, INC. DEPT. 5**
**9 S. Clinton Street**
**Chicago 6, Illinois**
**Please send free sample and descriptive literature.**

Name _____

Company _____

Address _____

City _____ Zone _____ State _____

## Special Events

The 3-day meeting included "coffee get-togethers" every morning for the ladies, as well as an all-day tour of the Henry Ford Museum and Greenfield Village with luncheon at the village on Tuesday, October 28, followed by an evening social hour for the conventioneers.

The inspection tour of the ACI headquarters building on Wednesday and the official dedication luncheon highlighted the special events of the convention. A tour of the Detroit Civic Center now under construction was guided by Harry Ellsberg and other staff members of Giffels and Rossetti, Inc.

## Local Committee Members

J. Gardner Martin headed the Detroit regional meeting committee with Warner Ohman acting as secretary; both are with Portland Cement Association, Lansing. John W. Winkworth, Winkworth Fuel and Supply Co., Detroit, served as treasurer of the committee and was in charge of financial arrangements for the meeting.

Elihu Geer, University of Detroit, headed the technical program committee and Charles Lyon of Master Builders Co. served as chairman of the publicity committee. Other Detroit area members serving as chairmen of the committees were: Lloyd Cheney, Wayne State University, in charge of educational contacts; W. G. Irmscher, Peerless Cement Corp., head of membership promotion; and Richard H. McClurg in charge of the ladies program. William A. Maples, American Concrete Institute, supervised registration.

---

## Speaking of vice-president Phil M. Ferguson's "past"

In giving a resumé of the accomplishments of Phil M. Ferguson in the October issue of the "News Letter" in connection with his nomination for 1959 president of ACI, we inadvertently published the fact that Professor Ferguson was a past president of the Texas Society of Professional Engineers. While he has been active in this organization he has not served in the capacity of president.

# SIKA-SEAL

## PROTECTS:
## FOUNDATIONS
## SEWAGE PLANTS
## WATER RESERVOIRS
## STEEL

Sika-Seal forms a tough, flexible and durable coating that provides an immediate vapor barrier when applied to structural materials.

It protects concrete, mortar, steel and other metallic and non-metallic surfaces. With Sika-Seal, your structure is protected from attacks by water, salt, dilute acids, mild alkalies and sea water —because Sika-Seal resists them all!

Moreover, Sika-Seal is a proven time and money saver since it can be painted directly on either damp or dry surfaces without delaying backfill operations.

In short, Sika-Seal is the ideal protection for wherever control of moisture, condensation and corrosive vapor is a must. For complete information on Sika-Seal and other Sika products for concrete and masonry, call or write.

26-24

**SIKA CHEMICAL CORPORATION**

PASSAIC · NEW JERSEY

DISTRICT OFFICES: ATLANTA • BOSTON • CHICAGO • DALLAS • DETROIT • NEW ORLEANS • PHILADELPHIA
PITTSBURGH • SALT LAKE CITY — DEALERS IN PRINCIPAL CITIES — AFFILIATES AROUND THE WORLD

**Architects:** Carson & Lundin (Bank portion only) Sydney Goldstone, & Kahn & Jacobs (rest of building)

**Structural Engineer:** Charles Mayer

**General Contractor:** George A. Fuller Co.

**Concrete Contractor:** Rizzi Construction Corporation

**Distributor:** Fireproof Products Company

In the new Textile Building . . .

# Short-span floor slabs reinforced with USS American Welded Wire Fabric

THE Textile Building, at 111 West 40th Street in Manhattan, was completed early this year. This office building reflects progressive design and construction techniques. As in so many other new buildings, a key construction feature is the short-span floors and roofs, reinforced with USS American Welded Wire Fabric.

*Short-span floors and roofs* consist of closely spaced beams supporting concrete slabs of minimum thickness. Such a design makes them light in weight, yet very strong. They are extremely economical, too, because less concrete, less time, and less labor are required to install short-span construction. But you couldn't have these savings without welded wire fabric reinforcement.

USS American Welded Wire Fabric makes short-span construction economical because the mesh can be draped continuously over supporting beams and higher working stresses are allowed than for mild-steel reinforcing bars. This is true in buildings designed in accordance with the New York City building code, and also anywhere the ACI Building Code is used.

*Specify USS American Welded Wire Fabric.* It's available in a wide variety of styles, sizes, lengths, widths, and finishes . . . in wire gauges from #7/0 to 16 and in longitudinal or transverse wire intervals of 2″ to 16″. Write for free catalog to American Steel & Wire, 614 Superior Ave., N.W., Cleveland 13, Ohio.

*USS and American are registered trademarks*

**American Steel & Wire Division of USS United States Steel**

Columbia-Geneva Steel Division, San Francisco, Pacific Coast Distributors
Tennessee Coal & Iron Division, Fairfield, Ala., Southern Distributors • United States Steel Export Company, Distributors Abroad

Remember, buyers will ask, "is it Reinforced"

# Reduce Weight with Voided Concrete Slabs

Middleville Road High School, Northport, L. I., N. Y.
Ellis Chingos Construction Corp., general contractors.
Severud, Elstad, & Kruger, structural engineers.
Ketcham, Gina & Sharp, architects.

***Form voids with low-cost***

**SONOCO**

**SONOVOID®**

**FIBRE TUBES**

***in concrete roof and floor slabs***

The circular concrete roof slab for the library building of the Middleville High School weighs less because it is voided with SONOVOID Fibre Tubes.

Weight reduction is only one of the many advantages of voided concrete construction. Voided roof and floor slabs have smooth undersides to which paint or plaster can be applied.

These and other features add up to the fact that concrete members, voided with low-cost SONOVOID Fibre Tubes, save time, money and materials.

Order in sizes from 2.25" O.D. to 36.9" O.D. Unless other lengths are specified, shipments will be made in standard 18' lengths. End closures available.

*See our catalog in Sweet's*

*For complete information and prices, write*

- HARTSVILLE, S. C.
- LA PUENTE, CALIF.
- MONTCLAIR, N. J.
- AKRON, INDIANA
- LONGVIEW, TEXAS
- ATLANTA, GA.
- BRANTFORD, ONT.
- MEXICO, D.F.

3096-A

**SONOCO**

Construction Products

SONOCO PRODUCTS COMPANY

# Positions and Projects

## Ultimate Strength Design Handbook to be work of new ACI technical committee

Total number of ACI technical committees in operation now climbs to 54 with the Institute Board of Direction approving the formation of Committee 340, Ultimate Strength Design Handbook. Edward Cohen of Ammann and Whitney, New York, will serve as chairman of the committee.

The initial mission is to develop a handbook containing data and design aids for reinforced concrete design by the ultimate strength method as authorized in "Building Code Requirements for Reinforced Concrete (ACI 318-56)." Emphasis will be on ultimate strength design for flexure and/or axial load. The handbook will be published separately as a special publication of the Institute.

## Change in title of ACI Committee 623

The Board of Direction has approved a change in title for ACI Committee 623. Previously called "Specifications and Practice for Foamed Concretes," the committee now assumes the new title "Cellular Concretes."

## Tuthill addresses ACI Southern California Chapter

Lewis H. Tuthill, California State Department of Water Resources, Sacramento, was guest speaker at the November 19 meeting of the ACI Southern California Chapter at the Rodger Young Auditorium, Los Angeles. Mr. Tuthill spoke on "What Do We Mean—Concrete Inspection?"

William A. Maples, ACI secretary-treasurer, who was in Los Angeles to wind up arrangements for the 55th annual convention to be held in Los Angeles, February 23-26, was special guest of the evening.

Chapter president, Ernst Maag, California Department of Public Works, Division of Architecture, Los Angeles, presided at a brief business meeting. Dewain R. Butler, Integrated Constructors and Engineers, Inc., served as program chairman for the meeting.

## Couse and Rogers appointed to national construction committee

Walter L. Couse and Lester C. Rogers have been reappointed to the committee on construction and civic development of the Chamber of Commerce of the United States for 1958-59. This committee studies major problems affecting the construction industry and urban development, and presents the business viewpoint on national legislative issues to congressional committees and government.

Mr. Couse is president of the firm of Walter L. Couse and Co., Detroit, and Mr. Rogers is president of Bates and Rogers Construction Co., Chicago. Both are ACI members of many years standing.

## Chisholm retires from Ontario Hydro Commission

T. H. Chisholm, for many years engineer-in-charge of the Chemical Research Department, recently retired from Ontario Hydro-Electric Power Commission, Toronto, after more than 40 years' service in the Research Division. At the time of his retirement he was leading a group of more than 30 chemists, chemical engineers, and technicians carrying on testing, analytic, investigation, and consulting work in a wide variety of technical fields.

Clyde H. Clark has been named to succeed Mr. Chisholm as engineer-in-charge, Chemical Research Department. He has been Mr. Chisholm's assistant for the past 10 years.

## Expanded Shale Clay and Slate Institute elects officers

The Expanded Shale Clay and Slate Institute has announced the election of the following officers for the ensuing year. President, Allan Taylor, Kentucky Light Aggregates, Inc., Louisville; 1st vice-president, Cedric Willson, Texas Industries, Inc., Ft. Worth, Tex.; 2nd vice-president, L. A. Thorssen, Consolidated Concrete Industries, Ltd., Calgary, Alta., Canada; and secretary, Lester Kennedy.

## Ruble appointed to ASCE executive committee

Emerson J. Ruble, research structural engineer, Association of American Railroads, Chicago, was appointed to the executive committee of the Structural Division, American Society of Civil Engineers, at the recent annual meeting of the society. Mr. Ruble will be the contact member with the ASCE Committee on Masonry and Reinforced Concrete and, consequently with those task committees under that administrative committee which are joint committees with ACI.

Mr. Ruble is currently a member of ACI Committee 115, Research; Committee 312, Plain and Reinforced Concrete Arches; and ACI-ASCE Committee 323, Prestressed Reinforced Concrete.

## Oklahoma State University conducts concrete proportioning course

In the interest of "continued education" in the field of concrete mix proportioning, an intensive 3-day program was recently held on the Oklahoma State University campus, Stillwater.

The course, sponsored by the School of Civil Engineering in cooperation with the Engineering and Industrial Extension, presented two recognized instructors, George Southworth, director of training for The Master Builders Co. of Cleveland, and Truman R. Jones, associate professor of civil engineering at Texas A & M College and also associate research engineer with the Texas Transportation Institute.

## CAP-AGC Joint Cooperative Committee organized

The formation of a national joint cooperative committee by the National Sand and Gravel Association, National Ready-Mixed Concrete Association, and Associated General Contractors of America was recently announced. It will be known as the Concrete and Aggregates Producers–Associated General Contractors Joint Cooperative Committee.

Leaders of the three organizations said the joint committee was especially needed because of important problems arising from the vast expansion of highway construction under the long range federal aid program and the steady increase in the dollar volume of construction activity in general.

The 16-man committee is composed of eight members representing the producers of concrete and aggregates, and eight representing the AGC. Robert Mitchell, Consolidated Rock Products Co., Los Angeles, and R. W. McKinney, Nacogdoches, Tex., are co-chairmen of the new group.

## Barrow-Aggee moves to new location

ACI member E. F. Harder, vice-president of Barrow-Agee Laboratories, Inc., announces new location of the organization in Chattanooga, Tenn. J. R. Mays, Jr., is president of the firm and G. L. Weaver is secretary.

## Ketchum and Konkel advance three firm members

Milo S. Ketchum and E. Vernon Konkel, partners in the firm of Ketchum and Konkel, consulting engineers, Denver, recently announced the advancement of three firm members, Ole F. Melleby, Eugene N. Catalano, and Earl F. Sorensen to the position of project engineer.

## Blast resistant civil defense center

The recently dedicated DuPage County civic defense center, two miles west of Wheaton, Ill., is designed to resist a blast overpressure of 30 psi, which is the design standard recommended by the Office of Civil and Defense Mobilization, according to Edward Grey Halstead, principal of Jensen and Halstead, architects for the project.

The floor of the building is approximately 2½ ft thick; the walls are 2 ft thick, and the ceiling is 3 ft thick. All are reinforced concrete. The ceiling was cast continuously in one day to eliminate construction joints. The concrete work was done by Imperial Flooring and Waterproofing Co. of Northlake, Ill. The structure measures 70 x 100 ft plus tunnels and electrical equipment vault.

# *Memorial Fund*

**The pronounced trend in recent years toward memorial funds is indicative of a general desire to place the emphasis less heavily on the emptiness left by the friend who has gone and more upon the fact that a life well lived, lives on.**

**Thus, the American Concrete Institute inaugurates a "Memorial Fund" for the commemoration of departed members. Contributions, comparable to what might otherwise have gone for flowers, can be made in the name of the deceased and an acknowledgment will be sent by the Institute to the member's family.**

**This month contributions to the fund have been received in memory of**

**Robert F. Blanks** **Newlin Morgan, Sr.**

## Joseph DiStasio

Joseph DiStasio, partner in DiStasio and Van Buren, New York consultants, lost his life on the Jersey Central commuter train which plunged through an open drawbridge into Newark Bay on Sept. 15, 1958.

Mr. DiStasio, graduate of Columbia University, was a recognized authority on reinforced concrete. He had written numerous technical papers dealing with the subject and received many awards for his outstanding contributions. In 1955 he received the Concrete Reinforcing Steel Institute Award for Outstanding Contributions to Reinforced Concrete. The award was made in recognition of his firm's pioneering and development of the flat plate system. In 1957 the Brooklyn Institute of Design and Construction named Mr. DiStasio the "Engineer of the Year."

His work included projects both in the United States and throughout the world. Some of the more recent projects were: structural design of buildings for new capitol of Brazil; 4000-ton U. S. Navy reinforced concrete floating drydock; and United Mine Workers' Hospitals in Kentucky and Tennessee.

Mr. DiStasio was active in a number of technical societies. He joined the American Concrete Institute in 1920, contributing frequently to the JOURNAL pages and participating in technical committee activities, particularly ACI Committee 318, Standard Building Code. At the time of his death, he was serving as chairman of Subcommittee 10, Committee 318, and was a member of Joint ACI-ASCE Committee 321.

The American Concrete Institute will make a posthumous award to Mr. DiStasio in February 1959 of the Henry L. Kennedy Award in recognition of outstanding contributions to the advancement of the objectives of the Institute.

## ROBERT F. BLANKS

**The Board of Direction of the American Concrete Institute regretfully takes note of the death of Robert F. Blanks, former Institute president and a long-time supporter of its activities. Both in his engineering career and in his professional society activities he made significant contributions to the knowledge of concrete and to means of making that knowledge available to the profession. He leaves an impressive record of outstanding accomplishments in engineering, business, and professional and technical societies. His enthusiasm was contagious and his straightforward approach to the problem at hand engendered confidence and support. His participation in the affairs of the Institute will be greatly missed.**

**—October 26, 1958**

# Honor Roll

## January 1—November 30, 1958

Is your state or country represented in the list of new members? If not, why don't you act promptly to recommend a friend for membership in the American Concrete Institute? Through your efforts, new members come into ACI and help to extend the scope of technical know-how of the Institute.

Blas Lamberti . . . . . 15
Joseph J. Waddell . . . . . 13
Phil M. Ferguson . . . . . 11
L. M. Legatski . . . . . 7
James A. McCarthy . . . . . 7
Martin J. Gutzwiller . . . . . 6
Ernst Maag . . . . . 6
H. C. Pfannkuche . . . . . 6
Jerome M. Raphael . . . . . 6
J. P. Thompson . . . . . 6
Carl E. Ekberg, Jr. . . . . 5
P. E. Ellen . . . . . 5
A. T. Hersey . . . . . 5
Samuel Hobbs . . . . . 5
Carl H. Koontz . . . . . 5
Douglas McHenry . . . . . 5
C. P. Siess . . . . . 5
Ernest L. Spencer . . . . . 5

Lower Cost
*Flexible*
POST TENSION CASING

U-100WP
(Wide Profile)
ECONOFLEX
Ask for Bulletin U-100WP

QUALITY...*All Metal*
Flexible Hose Products
*Universal*
METAL HOSE CO.
2101 S. Kedzie Ave., Chicago 23, Ill.

Robert P. Witt . . . . . 5
Moises Bendahan . . . . . 4
Celso A. Carbonell . . . . . 4
M. A. Craven . . . . . 4
Jaime de las Casas . . . . . 4
N. J. Everard . . . . . 4
E. L. Howard . . . . . 4
Frank E. Legg, Jr. . . . . 4
John B. Porter . . . . . 4
Walter H. Price . . . . . 4
C. H. Scholer . . . . . 4
Howard Simpson . . . . . 4
Clarence A. Walkwitz . . . . . 4
Luther E. Bell . . . . . 3
Vincent R. Cartelli . . . . . 3
Aleck S. Evans . . . . . 3
Richard W. Gunn . . . . . 3
Paul A. Hansen . . . . . 3
Robert B. Harris . . . . . 3
Kenneth M. Huber . . . . . 3
Thomas C. Kavanagh . . . . . 3
E. Richard Kummerle . . . . . 3
Daniel S. Ling . . . . . 3
Charles Macklin . . . . . 3
George A. Mansfield . . . . . 3
H. P. Mittet . . . . . 3
Francesco Sardella . . . . . 3
E. Schmid . . . . . 3
Otis D. Small . . . . . 3
O. C. Struthers . . . . . 3
T. W. Thomas . . . . . 3
Ruben Cano Vicario . . . . . 3
Ellis S. Vieser . . . . . 3
Byron P. Weintz . . . . . 3
William M. Avery . . . . . 2
Robert B. Banning . . . . . 2
Nicandro Jose Barboza . . . . . 2
O. R. Bell . . . . . 2
Massimiliano R. Berretti . . . . . 2
Carlos D. Bullock . . . . . 2
Antonio Cajigas . . . . . 2
Miles N. Clair . . . . . 2
W. L. Collier . . . . . 2
W. S. Cottingham . . . . . 2
Richard M. Dillon . . . . . 2
Lancelot A. Fekete . . . . . 2
Enrique Garcia-Reyes . . . . . 2
Thor Germundsson . . . . . 2
Louis A. Gottheil . . . . . 2
Einar Block Hansen . . . . . 2
Arch Hull . . . . . 2
Jorge Herrera Ibarra . . . . . 2
H. Alan Johnson . . . . . 2
Joe W. Kelly . . . . . 2
F. R. Khan . . . . . 2
R. G. King . . . . . 2
Carl O. Knop . . . . . 2
E. Vernon Konkel . . . . . 2
George Kurio . . . . . 2
James R. Libby . . . . . 2
Ciceron Hiedra Lopez . . . . . 2
William McGuire . . . . . 2
Bryant Mather . . . . . 2
Frank B. May . . . . . 2
Jorge L. Molina M. . . . . 2
Charles F. Moran . . . . . 2
I. E. Morris . . . . . 2
James A. Murray . . . . . 2

| | |
|---|---|
| D. F. Nash | 2 |
| Wendell H. Nedderman | 2 |
| Tatuo Nisigori | 2 |
| Ramiro Parada | 2 |
| Neftali Penaloza R. | 2 |
| J. L. Peterson | 2 |
| Victor M. Rada A. | 2 |
| Abdur Rahman S. Rasul | 2 |
| Clarence Rawhauser | 2 |
| Herbert Rusk | 2 |
| Leo L. Schaut | 2 |
| George Shervington | 2 |
| Joseph J. Shideler | 2 |
| George B. Southworth | 2 |
| Donald L. Strange-Boston | 2 |
| Ferruh Taskin | 2 |
| Sven Thaulow | 2 |
| Harry E. Thomas | 2 |
| I. L. Tyler | 2 |
| Oscar J. Vago | 2 |
| Jose Antonio Vila | 2 |
| Ivan A. Villamil | 2 |
| M. R. Vinayaka | 2 |
| Charles S. Whitney | 2 |
| Tsu-ming Yang | 2 |

# New Members

**The Board of Direction approved 56 Individual applications, 1 Corporation, 17 Juniors, and 15 Students making a total of 89 new members. Considering losses due to resignations and nonpayment of dues the total membership on Nov. 1, 1958, was 9652.**

### Individual

ADAMS, JOHN H., Chateauguay, Quebec (Sales Mgr., Chem. Spec., W. R. Grace & Co. of Canada, Ltd.)
APOCHALOW, GEORGE, Brooklyn, N. Y. (Struct. Designer, Madigan & Hyland, Cons. Engrs.)
BAKER, A. L. L., London, England (Prof., Imperial College)
BAKER, EDWARD E., Downey, Calif. (Designer, C. F. Braun & Co.)
BEGG, GEORGE, JR., Bethesda, Md. (Supv. Struct. Engr., The Public Bldg. Services)
BENTZ, CARL E., Columbus, Ohio (Cons. Arch., State of Ohio)
BIGOTT, A. W., Milwaukee, Wis. (Chf. Design Engr., Industrial Div., The T. L. Smith Co.)
BONILLA-GONZALEZ, CARLOS, San Felipe, Venezuela (Engr., Supt., D. O. F. C. A. de Construcciones)
BUNSUWAN, CHALERM, Bangkok, Thailand (Proj. Engr., People Irrigation Dept., Royal Irrigation Dept.)
BURMAN, DAVID C., Windsor, Ont., Canada (Design Engr., Giffels & Vallet of Canada)
CARR, BRIAN PATRICK, Auckland, New Zealand (Mgr., Vibrapac Block Factory, Winstone, Ltd.)
CARTER, GILBERT, JR., Burlington, N. J. (Staff Asst. to Plant Engr., New York Shipbuilding Corp.)
CARTER, HOWARD G., Monterey, Calif. (Cons. Civil & Struct. Engrg.)
CONNOR, EDMUND A., Melbourne, Fla. (Chf. Engr., W. B. Heath Engrg. & Surveying)
DOOLEN, DEWEY C., Springfield, Ill. (Gen. Supt., Evans & Lawrence Constr. Co.)
ELKIND, LAWRENCE L., Bronx, N. Y. (Sr. Engr., Struct., Bechtel Assocs.)
ESSWEIN, JOHN C., St. Louis, Mo. (Engr. in Chge., Bridges & Bldgs. Section, Board of Public Service)

**—and save these 5 ways . . .**

1. **Quicker setting**—less overtime finishing.
2. **High early strength**—faster form removal.
3. **Savings in protection time**—as much as 50%.
4. **Less delay** between operations.
5. **Added safety**—extra cold weather protection.

With a "Special Winterized" mix containing 2% of SOLVAY Calcium Chloride, heated water and aggregate, you keep close to warm weather schedules at any temperature—get concrete with 8 to 12% greater ultimate strength . . . more workability. This permits lower water-cement ratio, resulting in denser and more moisture- and wear-resistant concrete.

**Get the full story at no obligation—write for literature.**

**SOLVAY Calcium Chloride speeds but does not change the normal chemical action of portland cement.** Impartial tests by the National Bureau of Standards proved its advantages in cold weather concreting. This use of calcium chloride is recommended or approved by leading authorities, including American Concrete Institute and Portland Cement Association.

Allied Chemical

**SOLVAY PROCESS DIVISION**
**61 Broadway**
**New York 6, N. Y.**

***SOLVAY dealers and branch offices are located in major centers from coast to coast.***

FUGITT, L. C., Phoenix, Ariz. (Struct. Engr., Lescher & Mahoney)
FURR, HOWARD L., University, Miss. (Prof. of Civil Engrg., Univ. of Miss.)
GOEBEL, ROBERT V., Parma, Ohio (Gen. Contr.)
GUPTA, J. P., Nangal (Punjab), India (Exec. Engr., Nangal Fertilisers)
GUTHRIE, JOHN T., Chicago, Ill. (Field Engr., PCA)
HOELSCHER, F. W., Houston, Texas (Supv. of Concrete Inspection Dept., Southwestern Labs.)
HOSGOOD, FREDERICK JOHN, Johannesburg, South Africa (Gen. Mgr., Stuart B. Dickens Ltd.)
IMAN, MOHAMMED SHEET AL, Baghdad, Iraq (C. E.)
KLOZ, MILTON J., Whittier, Calif. (Sales Mgr., Standard Ready Mix Concrete)
LARSEN, HANS P., Montgomery, Ala. (Struct. Engr., Sherlock, Smith & Adams)
LEE, ROY R., Leamington, Ont., Canada (Struct. Design Engr., H. J. Heinz Co.)
LIMBADA, ISMAEL AHMAD, Karachi, Pakistan (Sr. C. E., Haydari Constr. Co., Ltd.)
LINDBERT, JOHN C., Jackson, Mich. (Sec., Treas., Gen. Mgr., Sager Concrete Products Co.)
LONG, SAMUEL B., JR., Philadelphia, Pa. (Partner & Cons. Struct. Engr., Kneas & Long)
MAHY, FRANK G., Whakatane, New Zealand (Bldg. Contr.)

## LOOKING AHEAD

Jan. 12-15, 1959—39th Annual National Concrete Masonry Association Convention and 11th Concrete Industries Exposition, Cleveland Public Auditorium, Cleveland, Ohio

Jan. 25-29, 1959—Associated Equipment Distributors, 40th Annual Meeting, Conrad Hilton Hotel, Chicago, Ill.

Jan. 27-30, 1959—National Crushed Stone Association, 42nd Annual Convention, Bal Harbour Area Hotel Group, Miami Beach, Fla.

Feb. 2-6, 1959—American Society for Testing Materials, Committee Week, William Penn Hotel, Pittsburgh

Feb. 15-19, 1959—National Sand and Gravel Association, 43rd Annual Convention, Hotel Roosevelt, New Orleans, La.

**Feb. 23-26, 1959—American Concrete Institute, 55th Annual Convention, Statler Hilton Hotel, Los Angeles, Calif.**

MAIER, LEONHARDT FRIEDRICH, Baltimore, Md. (Gen. Engr., Civil, J. George Stewart, Arch. of the Capitol)
MARROQUIN, CARLOS F., Mexico, D. F., Mexico (Asst. Gen. Dir., Cementos Anahuac, S. A.)
MAYHEW, WILLIAM, Jericho, L. I., N. Y. (Chf. Engr., Terry Contracting, Inc.)
MCALLISTER, HORACE A., Hicksville, N. Y. (Pres., Long Island Matls. Testing Lab.)
MCKENZIE, W. CAMERON, Vancouver, B. C., Canada (Cons. Engrg., Struct. & Civil Engrg., Choukalos, Woodburn, Hooley & McKenzie, Ltd.)
MURFITT, R. F., Calgary, Alta., Canada (Plant Engr., C. E., Peerless Rock Products Ltd.)
NARAYAN, JAGDISH, Mirzapur, U. P., India (C. E., Irrigation Dept. of the State of Uttar Pradesh, India)
PARONE, FRED J., Philadelphia, Pa. (Proj. Engr., Cochrane Corp.)
RAGA M., RAFAEL, Caracas, Venesuela (Dr. en Ingenieria Civil, Oficina Propia)
REYMERT, JAN FORBES, Oslo, Norway (C. E., A/S Prefab-Industri)
ROBSON, N. G., Rosemount, Montreal, Canada (Squad Leader in Struct. Dept., Shawinigan Engrg. Co.)
RONNEBERG, EARL F., Chicago, Ill. (Owner, N. Ronneberg Co.)
SENDA, SHINZABURO, Tokyo, Japan (Pres., Haneda Hume Pipe Co., Ltd.)
SNELLGROVE, GEORGE A., JR., Bergenfield, N. J. (Constr. Est., Fred J. Brotherton, Inc.)
SOFOS, NIKOLAOS, Athens, Greece ("DOMIKI," Inc.)
THUNEM, BERNARD ROY, Seattle, Wash. (C. E., Bruce C. Olsen)
WORKS, WILLIAM D., Chicago, Ill. (Archs. Repres. on Bldgs. under construction, Shaw, Metz & Dolio, Archs.)
WRIGHT, ROBERT ANGUS, Auckland, New Zealand (Asst. Struct. Engr., Thorpe, Cutter, Pickmere & Douglas)
YURIMA, JAA K., Toronto, Ont., Canada (Research Engr., Canada Iron Foundries, Ltd.)
ZALLEN, RUBIN M., Brighton, Mass. (Struct. Engr., Congdon, Gurney & Towle, Inc.)
ZAR, MAX, Chicago, Ill. (Assoc., Proj. Engr., Struct. Work, Sargent & Lundy)
ZEEVAERT, ADOLFO, Mexico, D. F., Mexico (Owner, Construcciones A. Zeevaert, S. A.)
ZEMSKY, CALVIN, Bayside, N. Y. (Proj. Engr., Terry Contracting, Inc.)
ZORZI, SILVANO, Milano, Italy (Struct. Engr. & Designer)

### Corporation

DIXIE FORM & STEEL CO., San Antonio, Texas (N. L. Stephens, Pres. & Gen. Mgr.)

### Junior

COLON, JOSE A. JIMENEZ, Ponce, Puerto Rico (Engr., Ponce Cement Corp.)
DOSHI, NALINKANT MOTICHAND, Rajkot, Bombay State, India (C. E. & Grad. Stud., Lehigh Univ.)
GENTRY, STEVEN CAVERHILL, Wellington, New Zealand (C. E., W. G. Morrison, Cons. Engr.)
HANSON, PAUL J., Lansing, Mich. (Struct. Designer, Clark R. Ackley, Arch.)
HITOMI, NOBUYUKI, Nagano-ken, Japan (The Kansai Electric Power Co., Inc.)
KHOURY, RAFAEL BIENVENIDO, Ciudad Trujillo, R. D. (C. E., Inspector, de la Seccion De mecania De Suelos De La Secretaria De Estado De Obras Publicas)
KOBAYASHI, MASAKI, Naganoken, Japan (The Kansai Electric Power Co., Inc.)
KONNER, ROBERT L., Baltimore, Md. (Instructor, The John Hopkins Univ.)
LINFANTE, ARTHUR A., JR., Newark, N. J. (Hwy. Engr., Howard, Needles, Tamman & Bergendoff)
MATEFFY, STEPHEN ALEXANDER, Sydney, NSW., Australia (Engr., Class 3, Dept. of Main Roads, NSW)
PERRY, ERVIN S., Baton Rouge, La. (Instructor, Civil Engrg., Southern Univ.)
PRAWEL, SHERWOOD P., JR., Buffalo, N. Y. (Asst. Prof. of Civil Engrg., Univ. of Buffalo)
REED, BARRY ERIC, Downsview, Ont., Canada (Struct. Engr., M. S. Yolles & Assocs.)
TAGLIABUE, CHARLES J., Cresskill, N. J. (Design Engr., Struct., Praeger-Kavanagh)
TEVAPALIN, SUPHOL, Bangkok, Thailand (Jr. Engr., Tech. Div., Royal Hwy. Dept.)

TURRENT VAZQUEZ, ERNESTO, Mexico, D. F., Mexico (C. E., Alen, S. A.)
WILLSON, JAMES M., Hagerstown, Md. (Resident Field Engr., PCA)

**Student**

CORLEY, WILLIAM GENE, Champaign, Ill. (Univ. of Ill.)
FAJARDO, ALVARO SILVA, Bogota, Colombia (Univ. Nacional Bogota)
GRONER, RICHARD S., Jefferson City, Mo. (Missouri School of Mines & Metallurgy)
GROUNI, HIDAYAT N., Lafayette, Ind. (School of Civil Engrg., Purdue Univ.)
KARAM D., SANTIAGO A., Caracas, Venezuela (Univ. Central De Venezuela)
KAYYAL, KASIM, Austin, Texas (Univ. of Texas)
KRAEMER, PAUL E., Lafayette, Ind. (Purdue Univ.)
LU, LE-WU, Bethlehem, Pa. (Fritz Lab., Lehigh Univ.)
MAGUIRE, FRANK J., III, New Haven, Conn. (Yale Univ.)
MARTIN, CHARLES W., Ames, Iowa (Iowa State College)
NAKAMURA, TSUNEYOSHI, Stanford, Calif. (Stanford Univ.)
RENSAA, ARTHUR JAMES, Berkeley, Calif. (Univ. of Calif.)
SHARMA, SATYA SAGAR, Champaign, Ill. (Univ. of Ill.)
TAN, FABER, Stockton, Calif. (College of the Pacific, School of Engrg.)
ZIMMERMAN, ROGER M., Boulder, Colo. (Univ. of Colorado)

## Los Angeles '59 Meeting

*Continued from p. 11*

A get-acquainted bruncheon will be held at the Statler Hilton on Tuesday morning, second day of the convention. Mrs. Charles Pankow will be chief hostess for the bruncheon with Mrs. Taylor Test and Mrs. Ray Cooley assisting. Hostesses for the fashion luncheon will be Mrs. Marvin Kudroff, Mrs. Test, and Mrs. Cooley.

## Burgess named assistant director of engineering

Woodrow L. Burgess has been appointed assistant director of engineering for The Master Builders Co., Cleveland. Mr. Burgess is well known for his work in the concrete field and the design and construction of large dams. He returned this August from Venezuela where, since October 1956, he was in charge of the design of two major dams for R. J. Tipton Associate Engineers, Inc.

## Wetzel named treasurer of San Antonio Portland Cement

The directors of San Antonio Portland Cement Co., San Antonio, Tex., have announced the election of Arthur J. Wetzel as treasurer of their organization.

PROFESSIONAL CARD

**JACKSON & MORELAND, Inc.**
**JACKSON & MORELAND INTERNATIONAL, Inc.**
*Engineers and Consultants*
Electrical—Mechanical—Structural
Design and Supervision of Construction for
Utility, Industrial and Atomic Projects
Surveys—Appraisals—Reports
Machine Design—Technical Publications
Boston New York

## Hews elected president of Yakima Cement Products

Richard J. Hews was recently elected president of the Yakima Cement Products Co., Yakima, Wash. Former president of the company, J. R. Sherman, retired June 1, 1958.

CONCRETE TESTERS
The World's Finest Low-Cost Precision Testers
For
CYLINDERS
CUBES
BLOCKS
BEAMS
PIPE
IF IT'S A CONCRETE TESTER YOU NEED-GET IN TOUCH WITH
FORNEY'S, Inc.
TESTER DIVISION
P. O. BOX 310 - NEW CASTLE, PA.

## ALPHABETICAL LIST OF ADVERTISERS

(Page Numbers refer to News Letter)

American-Marietta Company, Concrete Products Division....22

American Steel and Wire Division, United States Steel....30–31

Colorado Fuel and Iron Corporation, Clinton Division....25

Columbia-Southern Chemical Corporation....10

Concrete Reinforcing Steel Institute....6

Forney's, Inc....39

Jackson & Moreland, Inc....39

Johns-Manville....20

Lehigh Portland Cement Company....12

Master Builders Co., The....16

Sika Chemical Corporation....29

Solvay Process Division, Allied Chemical....37

Sonoco Products Company....32

Universal Metal Hose Co....36

Water Seals, Inc....28


The Institute assumes no responsibility for the claims of advertisers. The advertiser is made responsible in the belief that his place in the field will be determined by the public's ultimate measure of his exercise of that responsibility.

## Cleveland Engineering Society holds construction conference

Nine outstanding authorities in the field of construction appeared on the program of the 7th Annual Construction Conference sponsored by the Cleveland Engineering Society, November 12. Theme of this year's conference, which was sponsored by the construction division of the society, was "Construction Countdown."

Featured among the speakers were R. Buckminster Fuller, developer of the Geodesic Dome; Joseph Barnett, deputy assistant commissioner for engineering, Bureau of Public Roads; Truman R. Jones, Jr., Texas Agricultural & Mechanical College; O. H. Ammann and B. G. Anderson, Ammann & Whitney, New York; J. N. Mustard, Hydro-Electric Power Commission of Ontario; and D. C. Romick, Goodyear Aircraft Corp.

Highlight event of the program was an address by Keen Johnson, vice-president of Reynolds Metal Co. and former governor of Kentucky.

## Barnes and McNerney promoted by PCA

The appointment of Hugh D. Barnes to the position of supervisor of field promotion for the Portland Cement Association was recently announced by G. Donald Kennedy, PCA president. In this senior management position, Mr. Barnes will direct and have complete authority and responsibility for all operations of the association's 32 district and 6 regional offices.

John M. McNerney, district engineer for PCA Los Angeles office, will succeed Barnes as western regional manager.

# 155 Reasons Why...

What is the purpose of air entrainment in concrete?

How can durable aggregates be assured?

How do the requirements of placing affect the proportions of aggregates and cements?

. . . . The ACI CONCRETE PRIMER should be your guide to better concrete. The PRIMER develops in simple terms the principles governing concrete mixtures and presents a handy reference text for those who apply these principles to the production of permanent structures in concrete. Expanded to 72 pages—155 answers—the new pocket-size edition of the CONCRETE PRIMER brings developments of the past three decades into the question-and-answer format of this long-popular handbook.

**$0.50 TO ACI MEMBERS NONMEMBERS $1.00**

American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Michigan

Please send_____copies of the 1958 edition of the ACI Concrete Primer.
($0.50 for ACI members, $1.00 for nonmembers)

Check (or money order) enclosed for_____amount

Name.....

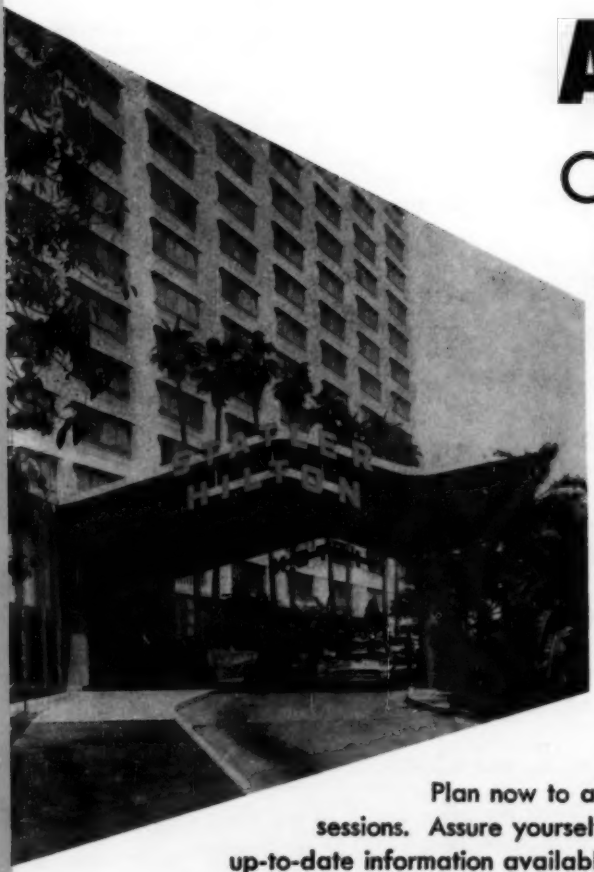
Address.....

City.....Zone.....State.....

# The 55TH Annual

# ACI

## Convention




### 8—Sessions—8

- Standards and Bylaws
- Construction
- Properties of Concrete
- Design and Analysis
- Concreting Materials
- Design Research
- Products and Precast Elements
- Research Session

Plan now to attend these "fact-filled" sessions. Assure yourself of the latest and most up-to-date information available in concrete and concrete construction. Make **your** reservations early.

## FEB. 23-26, 1959

## STATLER HILTON HOTEL, LOS ANGELES

n

8

uts

S